APPENDIX A: STRUCTURAL ANALYSIS AND DESIGN OF U-FRAME LOCK MONOLITHS

1. Introduction

This guidance is to be used by the structural engineer during the design of a U-frame lock monolith. A U-frame lock is a structure in which the base slab of the lock and the walls of the lock are monolithic. Therefore, a U-frame lock as discussed herein includes a W-frame lock structure. The advantages of the U-frame type of lock are the reduction in the volume of concrete in the walls, better seismic resistance, a reduced number of monoliths to design, a structure that is more readily dewatered, a possible reduction in pumping costs during dewatering due to less seepage, balancing of the loads applied to the monolith, and minimization of differential settlement and rotation of the walls with respect to the base.

1-1. Scope. This appendix includes technical guidance on structural analysis and design of U-frame lock monoliths. Planning and layout of navigation locks is covered in other guidance. Short excerpts of other guidance documents are repeated herein. Other guidance in this appendix includes definition of individual loads and load combinations; structural analysis methods and design assumptions; constructability; and serviceability. Additional guidance pertinent to U-frame lock design is contained in several other engineer regulations, engineer manuals, and engineer technical letters (as referenced in this document). Topics addressed by these other documents include strength design for reinforced concrete; seismic design; pile foundation design; and thermal-mechanical analysis of concrete. Such topics are addressed briefly herein; however, for details of these topics the engineer must see the referenced documents.

1-2. Applicability. This guidance should be used on any civil works project that contains a U-frame lock. The guidance provided herein can be used beginning with the reconnaissance phase of project design and should continue to be referenced through the preconstruction engineering and design phase and preparation of plans and specifications. The guidance may also be used as needed for engineering during construction.

1-3. References. References are included in Appendix B.

2. Design Planning

2-1. Coordination. Throughout the planning and design process, coordination within the design team is essential in achieving a quality design product. The design team should consist of representatives from construction/operations division, the cost sharing customer, planning, real estate, safety, cost engineering, and a representative from each of the engineering disciplines. Changes made to the structural configuration will often impact geotechnical, hydraulic, mechanical, and electrical engineers. Therefore, frequent communication with counterparts on a regular basis will facilitate identification of any problems that may have been created by the change. Coordination with higher authority is also necessary as described in ER 1110-2-1150.

2-2. Design sequence. Structural design of a monolith will be performed during various design phases. These are the reconnaissance phase, feasibility phase, preconstruction engineering and design (PED), and engineering during construction (EDC). The engineering requirements for each phase are defined in ER 1110-2-1150. Specific structural engineering responsibilities are defined in ETL 1110-8-13(FR), and general navigation lock design requirements can be found in other guidance. Each of these documents should be reviewed during the reconnaissance phase.

a. Reconnaissance report (RR). Analysis during the reconnaissance phase will usually be very limited; however, some analysis may be necessary to confirm the feasibility of the proposed plan. The initial decision to use a U-frame structure will be made during this phase. Since the plan presented in the RR is based largely on engineering judgment, it is important for an experienced structural engineer to be involved in this phase. The structural engineer must help develop a reasonable project configuration and the design cost and schedule for the next phase. Depth of the lock foundation should be carefully assessed during this phase since the depth can have a significant impact on construction cost.

b. Feasibility report (FR).

(1) Lock design during the feasibility phase (FP) must be sufficient to fully define the project scope and to develop an adequate baseline cost estimate with reasonable contingencies. The design will be presented in the engineering appendix to the FR. Also during the FP, the structural engineer must determine schedule and cost requirements for the remaining phases of the design. This information must be included in the Project Management Plan.

(2) Structural content in the engineering appendix will include a full definition of functional and technical design criteria, and initial analyses used to establish the basic geometry for the project. The suitability of a U-frame configuration must be firmly established during the FP. Decisions should be made on foundation type (soil or piles), chamber size, wall height and thickness, sill height, slab thickness, and foundation depth. The design team should also select types and sizes of guidewalls, filling and emptying systems, and closure systems for maintenance and emergencies. The structural engineer must be directly involved in this process, and sufficient analysis should be performed to verify these decisions.

c. Design memorandum (DM). Detailed design and analysis of the lock structure are documented in a DM. The DM is prepared either during PED or EDC phases. While design details may remain incomplete, the DM should contain an essentially complete design of the U-frame lock. During this design effort the structural engineer should accomplish the following: verify all design criteria; define all loads and load cases; select controlling load cases; develop a final pile layout if required; verify all member geometries; determine required concrete reinforcing steel; and calculate quantities for use in developing the cost estimate. This work should be thoroughly documented in the text and plates of the DM.

d. Plans and specifications (PS). Contract requirements are defined by the PS. The PS for the lock are usually prepared during the EDC phase since the PED phase ends with the initial construction contract for any other project feature. Structural engineering work for preparing the PS includes the following: lay out main reinforcement using quantities calculated for the DM; detail all secondary reinforcement; prepare and check contract drawings; develop and edit specifications; and calculate quantities for use in developing the government cost estimate.

2-3. Types of monoliths.

a. Background.

(1) As a result of the functional requirement for a lock to contain large tows, the chamber length parallel to flow can become very long. On a major waterway the chamber length can be up to 1,200 ft long. Consequently, it becomes necessary to incorporate monolith joints along the chamber length. The locations of these joints are used to define monoliths which have unique requirements towards the operability of the overall lock. Additional guidance with respect to locations, requirements, and lengths needed for the various types of monoliths is available in other documents.

(2) Monoliths that compose a lock can be categorized into five general groups. These five groups are:

(a) Intake/discharge monoliths.

(b) Gate monoliths.

(c) Culvert valve monoliths.

(d) Chamber monoliths.

(e) Other monoliths (e.g., guardwall monolith, bulkhead monolith).

(3) The complexity of design of these monoliths varies and the degree of complexity should be considered when assigning the design tasks for these various monoliths. Gate monoliths are the most difficult to design primarily due to the three-dimensional loading which is applied to these monoliths. Design of a gate monolith should be performed by a senior engineer. Intake/discharge monoliths and culvert valve monoliths can be difficult to design due to the fact that the geometries of these monoliths are difficult to evaluate in two dimensions. The chamber monolith is the simplest of the monoliths listed to design since it can usually be designed in two dimensions without concern about the out of plane direction. Design of a chamber monolith can be performed by a junior

engineer with the guidance of an experienced engineer. Other monoliths will generally be more difficult to design than a chamber monolith but not as difficult as a culvert valve monolith or intake/discharge monolith.

b. Intake/discharge monoliths. The intake and discharge monoliths are located at each end of the lock. The intake ports must be located in the upper pool while the discharge ports are located in the lower pool. Generally for a U-frame the intake/discharge monoliths will be U-frame monoliths, but it is not necessary if the applied loads are balanced on each wall. Placement of the manifolds towards the outside of the monolith will be advantageous in reducing congestion caused by the intersection of wall and base reinforcing if the manifolds are placed toward the inside of the monolith. However, site or hydraulic conditions may require that the discharge be placed on the inside of the monolith. Various port geometries and face locations have been used in the past and are generally established by the hydraulics engineer. The selection of the type of port should be made during the feasibility phase of a project, but the actual geometry will be determined for the design memorandum.

c. Gate monoliths. The gate monoliths are located at each end of the lock and house the gates used to let tows in and out of the lock chamber. In addition, these monoliths usually house the machinery used to actuate the gates. All anchor supports, bearings, and other embedded metals are contained within the walls and bases of these monoliths. Sills are frequently provided along the chamber base to establish draft requirements and provide a sealing surface to minimize flow between the gate and the concrete base. In the area of the sill, consideration should be given to including a recess in the base for silt deposits which would interfere with gate operation if recesses were not present. The gate monolith must also provide a recess to house the gate in its open position. Bulkhead slots are usually located upstream and downstream of the lock gate to allow for emergency and maintenance dewatering. These bulkhead slots may or may not be contained within the miter gate monolith.

d. Culvert valve monoliths. Culvert valve monoliths contain valves which control the filling and emptying of the lock chamber. A culvert valve monolith is located at the upstream and downstream end of the lock. The culvert valve is supported by anchoring into the monolith wall. The operating machinery for the culvert valve is also housed in the culvert valve monolith and is generally located at the top of the lock wall where steel frames are embedded in the concrete to secure the machinery. Bulkhead slots are provided for dewatering of the culvert valve recess for maintenance and inspection of the culvert valves. Since the culvert valve monolith generally lies between one of the massive gate monoliths and one of the more slender chamber monoliths, it is often used for a transition to align the culvert in the other two monoliths (see Figure A-1).

e. Chamber monoliths. The chamber monoliths are basically included to provide continuity of the lock chamber and culvert between the upstream and downstream culvert valve monoliths. Since the chamber monoliths are not required to support valves, gates, or operating equipment their walls are much thinner than the other monoliths. To save concrete, the outside edge of a chamber monolith can be tapered above the culvert. The chamber monoliths contain ports from the culvert to the lock chamber which are used for filling and emptying the lock. The spacing of these ports should be considered when determining the length of the chamber monolith. These monoliths will often contain a gallery near the top of the monolith which extends the length of the lock and is used to carry electrical wires and mechanical piping.

f. Other monoliths. Additional monoliths which may be included in a U-frame lock could be guardwall monoliths and bulkhead monoliths. A guardwall monolith will not always necessarily be a U-frame structure since it may only be needed on one side of the entrance to the lock. Typically, a guardwall monolith will be placed at the entrance of the lock to transition the area into the lock. A bulkhead monolith will become necessary if the location of the bulkheads is not contained within one of the other monoliths. If it is necessary for a lock to have a bulkhead monolith, it will often be of similar geometry to a chamber monolith. While a bulkhead monolith's geometry may be similar to a chamber monolith, its loading will not be since it will need to carry load in the upstream and downstream direction. Finally, any of the aforementioned monoliths can also act as bridge pier monoliths. When a bridge pier is located on a monolith, it can have an effect on the design of the monolith due to the loads transmitted to the monolith from the bridge pier. This is particularly true in active seismic areas.

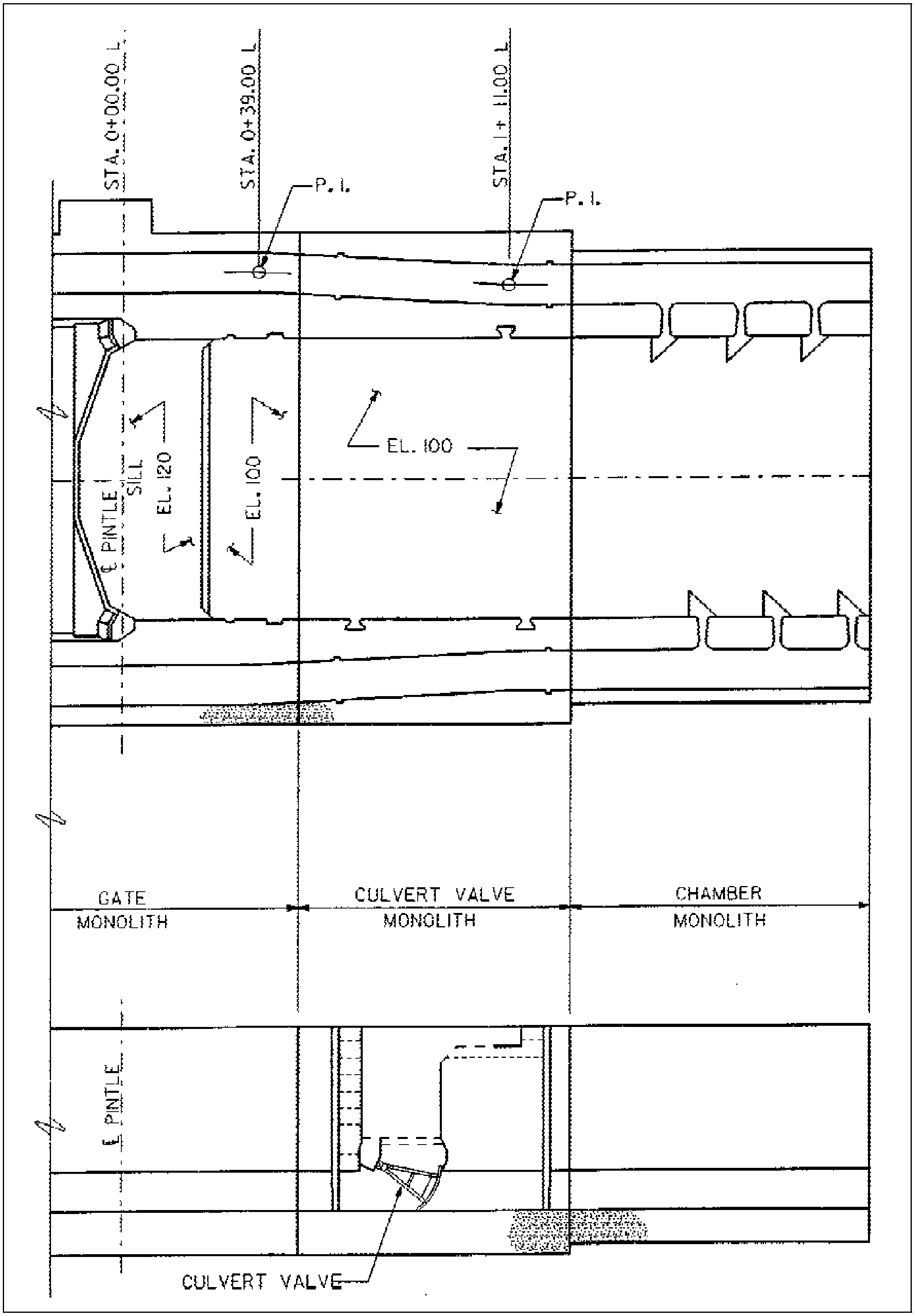


Figure A-1. Culvert valve transitions monolith. Plan view and elevation

2-4. Foundation alternatives. The decision whether to have a pile-founded or soil-founded U-frame lock must be based on numerous considerations: erodibility of foundation, potential for scour, factor of safety against flotation during dewatering of the lock, differential movements between monoliths, soil-bearing capacity, sliding stability for large unbalanced loads, the level of seismic activity, project layout, and cost. The type of foundation will affect the analysis methods and monolith geometry.

2-5. Two-dimensional versus three-dimensional behavior. U-frame lock monoliths can be categorized as two- or three-dimensional behavior. The behavior of a monolith is dependent upon both geometry and loading. Gate monoliths act three dimensionally due to loads on the gates which act in the longitudinal direction. Even though the behavior of a gate monolith may be three dimensional, it is possible to analyze a gate monolith by using a set of two-dimensional analyses to capture the three-dimensional behavior. Culvert valve and intake/discharge monoliths can be considered three-dimensional monoliths due to their geometry but generally two-dimensional approximations of these monoliths can be made which adequately capture their behavior for design purposes. Seismic analyses and nonlinear, incremental structural analyses of the culvert valve and intake/discharge monoliths should be three-dimensional analyses since the geometry of the structure has a much larger impact for these analyses than for static structural analyses. Chamber monoliths behave strictly as two-dimensional monoliths unless a loading in the longitudinal direction is placed on the monolith.

3. Design Criteria

3-1. General. The design criteria for navigation locks must be established during the feasibility phase of the design process. Criteria should cover stability, strength, serviceability, and foundation requirements. There are three categories of loadings defined in other guidance. These categories include usual, unusual, and extreme load conditions as described in paragraph 5-2. The separation of load conditions into these categories implys the nature, frequency, and consequence of the loading and also dictates the required factors of safety. The decrease in factor of safety allowed in all three categories of loading maintains limits that yield a linear elastic response of the structural elements. In some situations, however, nonlinear response could be acceptable depending upon the extent and duration of the response, redundancy of the structure, potential damage implications, and probability of occurrence. Decisions on acceptable nonlinear behavior must be coordinated with CECW-ED.

3-2. Stability. Stability criteria are applicable to soil-founded locks and include safety requirements against sliding, flotation, and overturning.

a. Sliding. Sliding stability is defined and calculation methods are shown in ETL 1110-2-256, and EM 1110-2-2502.

b. Flotation. Floatation stability is defined and calculation methods are shown in ETL 1110-2-307. Some provisions that are unique to U-frame locks can be found in other guidance.

c. Overturning. Overturning is not usually critical for U-frame lock monoliths because of the base width. However, bearing capacity at the outer edges of the structure is a concern. Stability with respect to resultant location is defined in EM-1110-2-2200 and EM 1110-2-2502. More information on resultant location calculations is found in paragraph 6-3. When performing resultant calculations it is important that unfactored loads be used. The resultant should fall within the middle third of the structure under usual loading conditions.

3-3. Strength. All components of a lock monolith must be able to resist all load conditions, including the reinforced concrete framing members, structural and miscellaneous steel, foundation piling, and foundation material.

a. Reinforced concrete. Detailed design guidance for reinforced concrete sections is covered in EM 1110-2-2104.

b. Steel structures. Design of structural steel, embedded metal, and miscellaneous steel should be based on EM 1110-2-2105. Design of major lock appurtenances such as miter gates, tainter gate valves, and associated machinery is covered under various other guidance publications including EM 1110-2-2703 and EM 1110-2-1610.

c. Foundation piling. Detailed design guidance for pile foundations is contained in EM 1110-2-2906. Further discussion is found in paragraph 6 below.

d. Foundation material. This information is generally determined by the geotechnical discipline and furnished to the structural engineer. Bearing strength for soils and methods for determining bearing strength based on field and laboratory data are described in EM 1110-1-1905.

3-4. Seismic design criteria. Seismic design criteria are defined in other guidance. Seismic load cases are generally considered unusual or extreme conditions and have reduced factors of safety associated with the criteria. The design that results from static analysis for usual and unusual conditions can be adequate for seismic response integrity. This is due to the use of damping effects, more sophisticated analyses, and different load factors in seismic analysis and design of reinforced concrete monoliths.

3-5. Serviceability. Serviceability requirements are unique among projects, and the designers are responsible for establishing these requirements. To establish serviceability requirements the design team should consider a number of aspects in the lock structure. Considerations should include minimization of concrete cracking, seepage and leaking, and reinforcement corrosion. Global deflections, settlement, and relative deflections are other primary concerns, especially those that affect mechanical interaction with the structure such as near the miter gate sill or valve locations. Additionally, maintenance, personnel access, and safety are important considerations for serviceability.

4. Loads

The most common loads on a U-frame lock are those due to the dead weight of the concrete structure, and those loads imposed from the soil and water which surround the structure. Additional loads from service gates, valves, bulkheads, emergency closure equipment, and operating machinery are also present. The stresses imposed by temporary loads must also be considered. Such loads are barge impact, ice, earthquake, wind, etc. Other loads such as surcharge and debris and ice loads may exist, depending upon the site-specific conditions. The following paragraphs provide a brief discussion on each of the commonly encountered loads and also provide guidance on how these loads are determined. Further information on these loads is contained in other guidance.

4-1. Dead loads. Dead loads consist of concrete and structural steel items such as miter gates, tainter valves, and emergency closure and maintenance bulkheads. The weight of the concrete structure is commonly the predominate force in the design of a U-frame lock. This load must be appropriately distributed so that its centroid coincides with the geometric centroid of the concrete item being analyzed. Effects of buoyancy on the concrete are accounted for separately as uplift forces described below.

4-2. Water. Water is either free standing or confined. Free-standing water refers to water above the soil, which is unaffected by either seepage or head loss. For example, water contained within the lock chamber, lock culverts, or outside the lock walls above any backfill, silt, ground surface, or concrete surface is free-standing water. This water produces pressures normal to any surface or plane which it contacts. For convenience, water pressures are considered such that the forces are either horizontal pressures or vertical weights. Confined water is that water which exists below the saturation line in any backfill and foundation material. This water produces pressure normal to any surface just as free-standing water except that the effect of seepage and head loss may need to be considered in determining the value of the pressure at any point. Water forces on monolith expansion or contraction joints resulting from broken waterstops must also be considered in analysis and design. The value of the horizontal pressure must equal the value of the vertical uplift pressure at any given point.

4-3. Uplift. The pressure of water creates forces acting upward on the bottom of the U-frame lock base. These uplift pressures are determined by multiplying the head of water above the bottom of the base by the density of water. The value of head used must include the effects of seepage from the upper pool to the lower pool. The rate of head loss must be determined for each project depending upon the permeability of the foundation, total head for the project, and presence of pressure relief systems (foundation drains). See Sherman (1968 and 1972) for additional details.

4-4. Soil pressures.

a. Vertical. The vertical weight of any soil acting on the structure is determined by multiplying the density of the soil by the volume of soil present. For soil above the water table, a moist density is normally used. For soil below the water table, a submerged (buoyant) density is used.

b. Lateral. The value of lateral soil pressure at any point is determined by multiplying the vertical weight of soil above the point (with the appropriate reduction in soil weight for when submerged) by the lateral pressure coefficient *k*. Observations from measured earth pressures on U-frame locks have shown that the coefficient of lateral earth pressure varies over a wide range along the height of the lock wall and also varies with time. Because of this variation, it is common practice to bracket the earth pressure by using an upper and a lower bound for *k*. These maximum and minimum values are then combined with the appropriate load cases for analysis in a conservative manner. Selection of *k* values for design should include knowledge gained from past instrumentation programs as well as results of classical solutions. Factors affecting *k* are flexibility of the wall, soil types, loading case, seismic condition, density of the backfill, and shear strength of the backfill. Examples of *k* values used vary from 0.2 to 1.0.

c. Silt. Since the load that the structure reacts to from earth pressures depends upon the depth of backfill, the possibility that silt could be deposited above the backfill must be taken into account. The amount of silt that may be deposited upon a structure is dependent upon the silt load the waterway is carrying, the water velocity at the location siltation is expected to occur, and the geometry of the structure. These factors vary for each project and must be considered when deciding upon the amount of deposited silt. For example, the lower Red River in Louisiana sees silt deposits of between 5 and 20 ft every time the river rises and falls. Silt loads are incorporated into the vertical and horizontal earth pressures by considering the silt as an additional soil layer above other existing soil layers. Normally silt deposited in the lock chamber is not considered as a load on the structure.

4-5. Drag. The downdrag force is a shear force acting downward along a vertical plane adjacent to or near a structure-to-soil interface (Ebeling, Duncan, and Clough 1990; Ebeling et al. 1992). Downdrag forces acting on the stem and culvert walls of U-frame locks are more difficult to characterize than are those for gravity walls founded on rock because of the interactions among the structure, the foundation, and the backfill. For guidance regarding downdrag, reference should be made to the results discussed in the report on the Port Allen and Old River Locks (Clough and Duncan 1969) as well as the report on Red River Lock and Dam No. 1 (Ebeling et al. 1993). These studies show the extremes with respect to downdrag which have been computed to date. Given the state of practice, a complete soil-structure interaction (SSI) analysis is the most reliable procedure available for estimating downdrag forces on soil-founded U-frame locks because the more compressible the foundation is, the greater the need for SSI analyses to determine the values to be assigned for downdrag forces. The finite element method of analysis is used in this type of analysis to compute the stresses and displacements for both the structure and the backfill. The finite element program SOILSTRUCT has the capabilities for performing a complete SSI analysis to obtain downdrag forces and has been used successfully on numerous projects, including those cited in this section (Ebeling, Peters, and Clough 1992).

4-6. Foundation pressure. Foundation pressure is the response of the soil to the force of the structure placed upon the soil. To determine the foundation pressure, see paragraph 6, Foundation Analysis.

4-7. Impact. Impact loads from barges striking the lock walls should not be considered in the overall design of the monolith, but should be considered in localized areas. These loads are covered in ETL 1110-2-338. The impact loads on the lock walls are generally less severe than those on more exposed components of a navigation project since the angle of impact is limited in the lock, and speeds are generally much slower in the lock. The magnitude of these forces is dependent upon the size of the tow and barge that the lock can accommodate and therefore is project-dependent. Loads transferred through the lock gates due to impacts on the gates should be considered. Guidance on values to be applied to gate impacts is included in EM 1110-2-2703 and EM 1110-2-2105.

4-8. Hawser. Hawser loads should not be considered in the overall design of the monolith, but should be considered in the design of the upper part of the lock wall. Hawser loads are the forces generated resisting the inertial forces of moving barges. Hawser loads act in the opposite direction to impact loads. These loads are covered in other guidance. For additional information, see ETL 1110-2-247.

4-9. Seismic loads. There are two types of seismic analyses, psuedostatic and dynamic. The psuedostatic analysis is used in low seismic zones and with low peak ground accelerations (PGA). The dynamic analysis is used in higher seismic zones and with higher PGA's. The extent of seismic analysis required depends upon the results of the seismological site investigation. Detailed requirements on the seismic design of U-frame locks are covered in other guidance.

4-10. Gate reactions. Any force resulting from the dead weight of lock gates or culvert valves and forces transferred from the gates due to hydrostatic pressure acting on the gates should be considered in the design of the gatebay or valve monoliths. EM 1110-2-2703 describes how these forces should be determined and applied to the monolith.

4-11. Thermal. Thermal loads are caused by volumetric changes caused by changes in temperature, temperature gradients through sections of the concrete structure, geometric discontinuities in the structure (e.g. culverts), and external restraints (e.g. piles). Thermal loads must be evaluated in a U-frame lock structure because large member thicknesses prevent the heat generated from hydration from being dissipated as quickly as it is generated and therefore the temperatures within the structure rise. Effects of thermal loads can be evaluated through a nonlinear, incremental structural analysis as described in paragraph 7-5.

4-12. Cofferdam tie-in. The forces exerted on the structure by cofferdams should be incorporated into the appropriate monoliths. The forces in sheetpile interlocks as well as any horizontal loads and drag loads from the fill within the sheetpile cells should be added to the appropriate monolith.

4-13. Sheetpile cutoff reaction. The sheetpile cutoff reactions on U-frame monoliths are generally small and neglected. For further information, see other guidance.

4-14. Localized loads. Loads from equipment and appurtenant items are discussed in other guidance. These loads must be carried by the lock monolith. Localized loads normally do not control the structure design or overall stability, but may control the design in localized areas. Examples of this type of load include the horizontal thrust from the miter gate bull gear support frame, the emergency bulkhead crane pedestal, support columns for control houses, tainter valve trunnion anchorages, miter gate gudgeon pin anchorages, miter gate pintle bases, miter gate latches, emergency bulkheads and maintenance bulkheads, emergency bulkheads lowering carriage machinery, jacking forces on gatebay monoliths due to gate diagonals tensioning operations, area lighting towers, etc. In the case of jacking forces on gatebays due to gate diagonals tensioning operations, anchors and/or jacking points should be provided for in the design of the gatebays to ensure that sufficient means are available to tension miter gate diagonals.

4-15. Other loads. Wind loads are relatively small and should be neglected. Ice loads on lock walls are not ordinarily included in the structural design. However, approach walls and mooring facilities, particularly those items in the upper approach, are sometimes subjected to moving ice and the effects should be accounted for. For isolated installations where ice conditions are severe, and the ice sheet is short and can be restrained or wedged between structures, its magnitude should be estimated, with consideration given to availability of records of ice conditions. It is recommended that an impact pressure of not more than 5,000 lb/sq ft be applied to the contact surface of the structure, based on the expected ice thickness. In the United States the ice thickness assumed for design normally will not exceed 2 ft. Ice pressure should be applied at the upper pool elevation. For further information, see EM 1110-2-1612, ETL 1110-2-295, and ETL 1110-2-320. Superstructure loads include the reactions to control houses and access bridge spans. Miscellaneous loads include large temporary surcharge loads and mobile equipment loads. Typical items may include cranes for maintenance and placement of stoplogs, construction equipment used for concrete placement, etc.

5. Load Cases

5-1. General. Load cases are combinations of the various loads described in paragraph 4. The forces in the load cases are factored or unfactored depending upon the analysis being performed. The load factors used depend upon the type of load, certainty of magnitude of load, and frequency of load being applied.

5-2. Categories of load cases. The structure during its life will be subjected to many differing loads. The severity of these loads and the frequency of their occurrence along with the consequences of the structure being damaged lead one to grouping the load cases into three different categories: usual, unusual, and extreme. A usual load case is one that affects the structure for extended periods on a recurring basis. Such load cases permit no reduced load factors to the structure components. An unusual load case is one that the structure sees occasionally and/or for short periods. Such load cases have minor reductions to load factors on the structure components. An extreme load case is one that might happen only once or twice to the structure. Such load cases have major reductions to the load factors on the structure components.

5-3. Load combinations. Load cases are formed by combining the individual loads together. Usually the controlling load cases will be those with the greatest vertical or lateral loads. However, some cases may control that do not have the highest vertical or lateral load but for which the combination of lateral and vertical loads is more severe. In addition, for monoliths subject to dewatering, the dewatered case may require that additional means be provided to resist uplift or that limits be placed upon the maximum pool elevations at which the monolith may be dewatered. Both maximum and minimum coefficients of lateral soil pressure can be used in the factored stress analysis to bracket the actual lateral soil pressures. Also, both uniform and stepped bearing pressure distributions are used in the factored stress analysis to bracket the actual base pressure distribution. By combining the above loads with the varying foundation pressure distributions and limiting lateral soil pressure distributions, the number of possible loading combinations soon becomes astronomical. All of these combinations need not be analyzed. Sensitivity studies, engineering judgment, and other rational methods should be used to select a reasonable number of cases to analyze. It is important to note that for pile-founded structures, some load cases may not be critical for the pile foundation design but could control the concrete monolith design. All load cases used on pile-founded structures should be analyzed after the final pile layout has been developed so that proper pile forces are included in the concrete monolith design.

5-4. Application of load factors. Analysis with unfactored (service) loads is used for foundation design and can be performed for the ultimate strength design of reinforced concrete. For frame analysis with service loads, internal member reaction results are factored for member design. Application of appropriate load factors is simplified when internal member results are categorized as dead and live loads. Alternatively, analysis can be performed with factored loads which directly yields internal member reactions for member design. Analysis with factored loads causes the points of inflection to shift and the foundation pressure distribution to change when compared with an analysis with service loads.

6. Foundation Analysis

6-1. Determination of type of foundation. The selection of the type of foundation is probably the most critical aspect of the design of a U-frame lock because of cost considerations and the overall behavior of the structure. Since this decision will have a significant impact on the project cost, the determination of the type of foundation should be made in the feasibility phase of the project. A thorough subsurface investigation and testing program should be undertaken to define the soil strengths and parameters. A soil foundation is usually more economical if special measures (deeper excavation, elaborate pressure relief system, etc.) are not required. The soil foundation has to be able to satisfy stability requirements for sliding and overturning, as well as resisting uplift (flotation) and earthquake forces. Included in evaluating soil-founded versus a pile-founded lock should be the consideration of differential settlements between monoliths. If a soil foundation is not feasible because of site conditions, then a pile foundation is required. When considering a pile foundation, all types of piles should be considered and the most feasible and economical types of piles should be chosen based on strength, geotechnical conditions, availability of material, and construction limitations. In order to make these comparisons and the comparison to the soil foundation alternative, pile quantities should be computed based on assumed lateral and vertical pile capacities and the minimum pile spacing that is expected, taking into consideration the fact that the density of piles may need to be higher in some areas of the structure than others. This quantity computation should be performed on one of the more massive monoliths and on one of the chamber monoliths, and the results should then be extrapolated out for the entire lock. If this initial comparison shows the pile foundation to be more economical or approximately the same as the soil foundation, then the designer should proceed with a more detailed analysis of the layout using the most economical type of pile. A rigid base analysis can be performed using computer program CPGA (Case Pile Group Analysis) (Hartman et al. 1989) for this purpose, or if a two-dimensional monolith is being evaluated, a flexible base analysis can be performed using CWFRAM. The final decision, which will be based on cost, can be made using the refined pile layout compared with a soil founded configuration.

6-2. Pile founded.

a. General. The pile foundation design development should follow a procedure that conforms to the normal submittal phases for civil works design projects. During the feasibility phase of the project, the determination for use of a pile foundation should be made, the most economical type of pile to use must be decided, and an approximate cost of the foundation should be projected. The actual detailed design of the pile foundations should occur during the preparation of design memorandums for the project. The final pile layouts for all monoliths should be developed so that only minor refinements and addition of details are required during the development of plans and specifications.

b. Types of piles. There are many different types of piling that can be used for a U-frame lock foundation, each with its advantages and disadvantages. Common types of piling include: steel H-piles, steel pipe piles, precast concrete piles, cast in place concrete piles, mandrel driven piles, and timber piles. For a detailed discussion of the types of piles and how to evaluate them, see EM 1110-2-2906.

c. Initial pile layout. Determination of the initial pile layout should be made in the feasibility stage. Preliminary layouts for costing purposes can be accomplished by using conservative lateral and vertical capabilities for a single pile and applying these values to resist the total lateral and vertical loads for the worst load cases. This gives a very rough idea of the total quantity of piling required. Piles should be located in grid patterns relative to concentrations in foundation forces, geotechnical considerations, and pile-driving tolerances. The grid should be established so that no pile interferes with another, and such that no interference occurs among pilings under adjacent monoliths or with sheetpile cutoffs. Computer program CPGI (Pile Group Interference Check, CASE computer program X0086) can be used to determine interferences. The grid should also consider the effects of close pile spacing on design criteria, particularly for friction piles. Generally speaking, for U-frame locks, greater concentrations of piling should be located beneath the heavier portions of the monolith such as the lock walls and less dense concentrations beneath lighter areas like the chamber floor. Also, tension piles may be needed at the center of the chamber floor to provide resistance against uplift during maintenance or other dewatered conditions. Tension anchors could also be used in this regard, and the use of drains can help reduce uplift forces. Preliminary pile analyses at this stage could be performed using CPGA (Hartman et al. 1989) which assumes an infinitely rigid base and allows for two- or three-dimensional analysis. For U-frame locks, the rigid base assumption is not necessarily correct but is satisfactory for determination of preliminary pile quantities for costing purposes. Preliminary pile layouts should be developed for the major types of monoliths that comprise the lock so that accurate costs can be obtained. Calculation of pile loads and some refinement in the layout can be accomplished using CWFRAME (Jordan and Dawkins 1990). This program analyzes two-dimensional plane sections through the lock and accounts for the flexibility in the base of the lock structure. This will give a more accurate distribution of pile forces for a U-frame lock than will CPGA (Hartman et al. 1989). However, three-dimensional effects must be accounted for independently and added to the two-dimensional CWFRAME (Jordan and Dawkins 1990) results. This flexible base behavior verification is particularly important for monoliths that may require tension piling under the lock floor to resist uplift during dewatering.

d. Final pile layout and analysis. If the pile layouts were properly developed during the feasibility phase of the project, they can be used as a good starting point for development of the final layouts. Lastly, after an acceptable pile layout is determined based on assumed critical cases, all load cases should be checked for effects on the pile layout and the base slab bending.

e. Rigid versus flexible base. The designer should determine the relative rigidity of the lock base with respect to the pile foundation. This can be accomplished by running some simple parametric studies in which the pile forces for a simplified flexural model are compared with the rigid base results. For U-frame locks, most monoliths should be analyzed as flexible base structures unless it can be shown that the rigid base results closely approximate the flexible base. Initial design estimates could consider the base rigid, and the preliminary analysis can be performed using a rigid base analysis tool, such as the computer program CPGA (Hartman et al. 1989). This program performs the pile analysis for two- and three-dimensional loading conditions. For subsequent foundation design and any structural concrete design, the pile cap should be treated as a flexible base. Therefore, the pile foundation and structural analysis must be performed using a program that will consider the internal stiffness relationship of the structure. The same flexible base analysis can be used to analyze the piles and the concrete structure. This is possible using the computer program CWFRAME (Jordan and Dawkins 1990) or other frame analysis or finite element applications which include pile elements. A complete SSI finite element model can be useful for this analysis, though it is usually much more complicated.

f. Pile stiffness coefficients. Before beginning any detailed pile analysis, the pile stiffness coefficients should be determined by performing single pile analyses based on available soil data or previous pile tests with similar soils and conditions. These coefficients are essentially linear springs that approximate the nonlinear behavior of the soil-pile foundation. Normally, it is desirable to perform a parametric analysis where the bounds in variability of the coefficients can be captured. Once determined, these coefficients are input to the various programs chosen for the pile group analysis. See EM 1110-2-2906 for a further discussion of stiffness coefficients.

g. Lateral load resistance. Lateral load resistance in pile foundations is dependent upon the pile type, strong axis orientation and batter angle, and on the assumed or experimental lateral subgrade moduli used in design.

(1) Pile orientation. Lateral loads are most efficiently resisted by battered piles. However, battered piles are more difficult to drive and result in a more complicated layout to design and construct. Additionally, battered piles tend to dramatically change the pile force distribution. If the lateral loads are not significant, the designer should consider using all vertical piles. If the lateral load is significant, piles with unequal stiffnesses about the orthogonal axes (H-piles for example) can be turned to increase stiffness in the direction of the load or they can be battered. The preliminary batter slope and number of battered piles can be determined by using force vectors or similar methods. Capacities, limitations, and suggestions for use of battered piles are defined in EM 1110-2-2906. In using battered piles, consideration must be given to geometric constraints from adjacent pile-founded monoliths and sheetpile cutoffs. These constraints can be assessed using the computer program CPGI (Pile Group Interference Check, CASE computer program X0086). Generally, piles from one monolith should not extend into the area beneath an adjacent monolith because of the possibility of interference.

(2) Pile head fixity. If it is not practical to use battered piles to resist the lateral loads because of geometric constraints, all vertical piles may still be a possible solution, but lateral deflections may become critical. If lateral deflections are too high using vertical piles with a pinned condition at the pile head, the pile may be embedded deeper and analyzed as fixed at the pile cap. Pile head fixity is discussed in EM 1110-2-2906. Refer to Castella (1984) for more information on pile head fixity.

(3) Lateral subgrade moduli. When an acceptable initial layout is achieved based on pile forces and stresses, a comparison of calculated pile head deflections to those seen in test results or assumed in the pile stiffness coefficient analysis must be made. Since the pile/soil stiffness degrades with deflection, the calculated deflections seen in analyses should compare with the deflections assumed or generated in the selection of the pile stiffness coefficients.

6-3. Soil founded. The analysis of the foundation includes checking for resultant location, sliding, uplift (flotation), differential settlement, and bearing failure. One of the single most important elements in the design of a soil-founded U-frame lock is the assumption regarding the distribution of the effective base pressure.

a. Pressure distribution. There are two basic approaches used in determining the distribution of base pressures. One is a soil spring approach and the other is an assumed pressure distribution. The spring method is discussed in paragraph 7-3*e*. The assumed pressure distribution approaches are a uniform distribution and a stepped distribution with appropriate corrections for eccentric loading. The methods presented have been derived from analysis of instrumentation data where base pressures were measured and compared with conventional calculations. A tool available to compute base pressures is the CASE computer program 3DSAD (Tracy and Kling 1982).

The uniform pressure distribution for concentric loading is based upon the assumption that the base slab of the monolith behaves as a rigid base. To compute the pressure distribution, the sum of all vertical forces acting on the base of the monolith is distributed equally across the monolith bottom. An example of this computation is given in Figure A-2. This uniform pressure is modified to account for loads eccentric to the centroid of the monolith base. An example of this computation is given in Figure A-3. The stepped pressure distribution is an approximation based upon observed data from instrumented U-frame locks founded on soil. In this distribution, the pressure beneath the lock wall is increased to a set percentage of the pressure beneath the remainder of the monolith. These two pressures are modified proportionately until the total pressure equals the sum of the vertical forces. Observed data from Port Allen Lock suggest the amount of increase should be 75 percent. An example of this computation is given in Figure A-2. This stepped base pressure is modified to account for loads eccentric to the centroid of the monolith base. An example of this computation is given in Figure A-3. For further information on the amount of increase to use, see Sherman (1968) and (1972).

b. Location of resultant analysis. A resultant location analysis using unfactored loads should be performed on each two-dimensional and three-dimensional monolith. The analysis consists of determining the location of the resultant of all loading in relation to the kern of the monolith base. The resultant location for usual load cases should be the middle third of the base. The resultant location for the unusual load cases should be the middle half of the base. The resultant location for the extreme cases should be within the base. Usually the location of the resultant is not a problem.

c. Sliding analysis. A sliding analysis using unfactored loads should be performed in accordance with ETL 1110-2-256. The CASE computer program CSLIDE can perform this analysis (Pace 1987).

d. Bearing analysis. A bearing pressure analysis should be performed using unfactored loads. The foundation capacity should be developed taking into account such items as soil type and stratification. The computed bearing pressures must be less than the foundation capacity.

e. Flotation. A flotation analysis using unfactored loads should be performed on each monolith that can be dewatered. For further information, see ETL 1110-2-307. Drag loads will not be used to resist uplift (flotation) due to the varying nature of drag loads. If insufficient capacity exists to keep the monolith from floating, the monolith can be held down with anchors, heels, or more concrete mass, or improved foundation drainage systems can be added to the monolith.

f. Differential settlement. Differential settlement occurs between adjacent monoliths due to the difference in size and weight of the monoliths as well as differing foundation conditions beneath each monolith. Differential settlement should be held to the practical minimum possible. There are several ways to handle this problem, including use of keys, dowels, and construction sequencing. Keys can be formed between adjacent monoliths. Dowels can be added between the base slabs of adjacent monoliths. The construction sequence for adjacent monoliths can be specified such that the heavier monolith is partially placed prior to placement of the lighter monolith. The magnitude of the forces being carried by dowels or keys is difficult to predict, but the designer must try to account for these forces by some rational method.

7. Structural Analysis

Once the design criteria have been established, all reasonable load cases have been identified, and the initial foundation parameters have been established, the analysis of the structure may be performed. The structural analysis is necessary for ensuring that the wall and slab thicknesses are sufficient and for determining the reinforcement requirements of the structure. Before performing an analysis of a U-frame lock, the designer must decide whether each monolith behaves in a two-dimensional or a three-dimensional manner. The method of analysis must also be selected, which can be a frame analysis, a finite element analysis, hand calculations, or a combination of these. These decisions are based on experience and good engineering judgment. A parametric study which bounds the extremes of behavior of a structure can also be used as a tool to ensure adequacy of a structure. As a result of these analyses, the designer can then determine final member sizes and reinforcement.

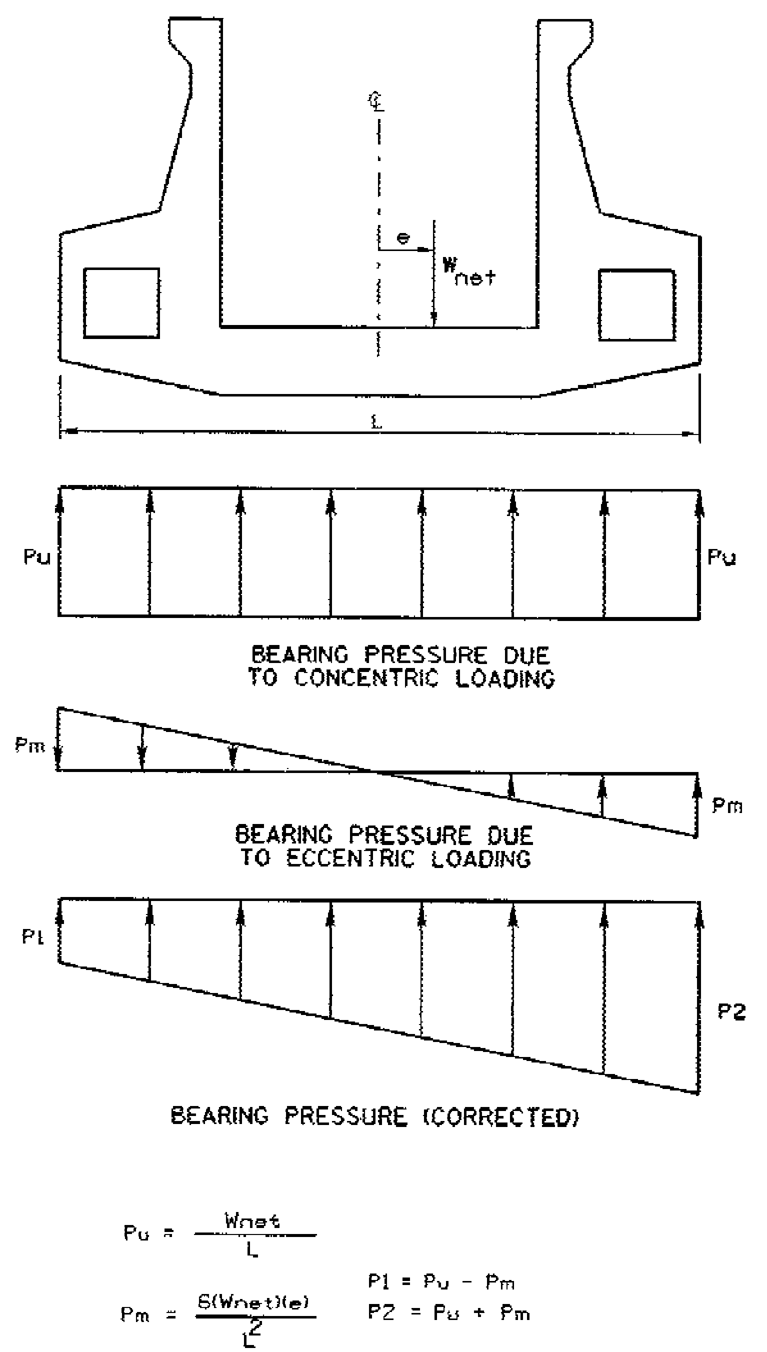
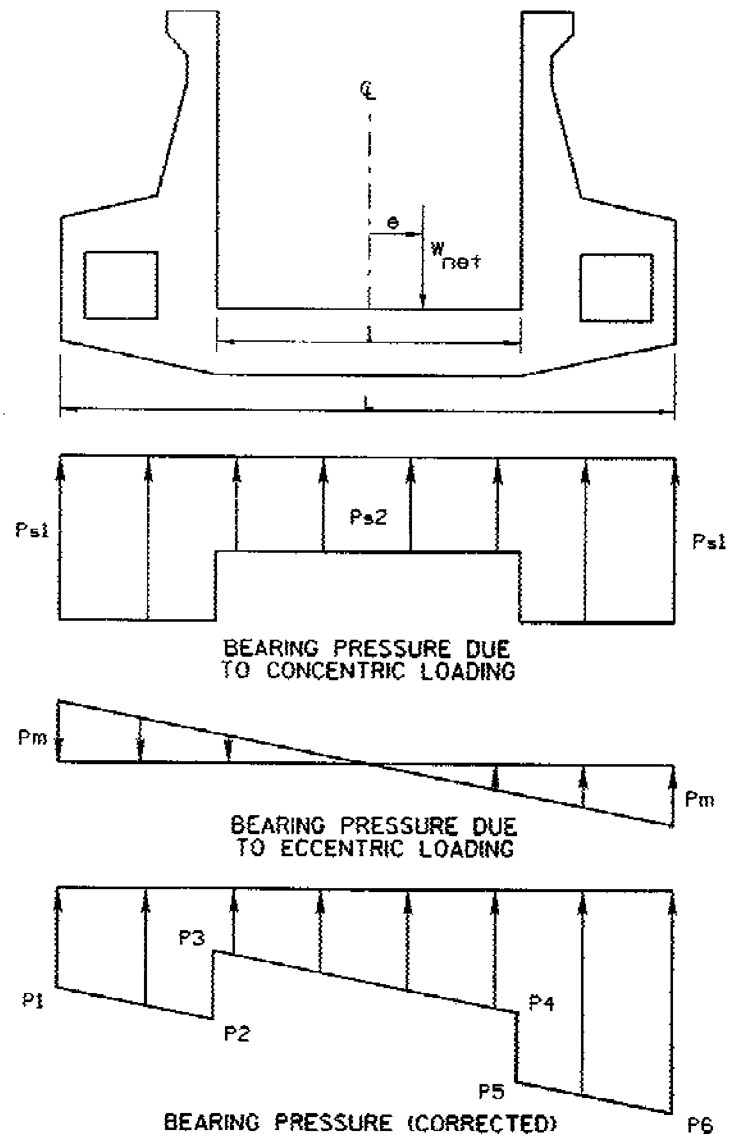


Figure A-2. Bearing pressure - uniform



$$Ps1 = (1+NR)Ps2$$

$$Ps1(L-l)+Ps2(l)=PuL = Wnet$$

$$Ps1 = \frac{(1+NR)Wnet}{(1+NR)(L)-(NR)(l)}$$

$$Ps2 = \frac{Wnet}{(1+NR)(L)-(NR)(l)}$$

$$Pm = \frac{6(Wnet)(e)}{L^2}$$

$$P1 = Ps1 - Pm$$

$$P2 = Ps1 - Pm\left(\frac{l}{L}\right)$$

$$P3 = Ps2 - Pm\left(\frac{l}{L}\right)$$

$$P4 = Ps2 + Pm\left(\frac{l}{L}\right)$$

$$P5 = Ps1 + Pm\left(\frac{l}{L}\right)$$

$$P6 = Ps1 + Pm$$

NR = % INCREASE BENEATH WALL ($0 \leq NR \leq 1.0$)

Pu = UNIFORM BEARING PRESSURE

Figure A-3. Bearing pressure - stepped

7-1. Two-dimensional monoliths.

a. Lock monoliths that meet the following requirements can be considered two dimensional for analysis purposes:

(1) The cross section of the monolith, transverse to the lock centerline, is constant or nearly constant.

(2) Loads acting on the monolith do not cause significant overturning of the monolith in the direction parallel to the lock centerline.

(3) Loads acting on the monolith do not cause torsion of the monolith. Torsion is considered to be rotation about a vertical axis through the center of gravity of the monolith.

Monoliths which meet these requirements can be analyzed using a typical strip. Generally, chamber monoliths, and for some cases culvert valve monoliths and intake/discharge, can be considered to act two dimensionally. In certain cases the loading and geometry may be such that some of the above requirements are not completely satisfied but a two-dimensional analysis may still be used to accurately model portions of the monolith.

b. Much of the analysis of U-frame locks can be performed using frame analysis methods as described below. For two-dimensional frame analysis no additional loads from adjacent monoliths/strips should be applied. Should investigation of a monolith indicate that a frame analysis is not adequate for analysis of the structure (e.g., a monolith which has walls with a low member length-to-depth ratio), a finite element analysis should be performed. Typically, for a finite element analysis of a two-dimensional structure, a plane strain analysis should be performed. Another application of finite element analysis for two-dimensional analysis is to calibrate and verify the results from frame analyses.

7-2. Three-dimensional monoliths.

a. Lock monoliths which do not meet the requirements for two-dimensional monoliths must be analyzed as three-dimensional monoliths. Gate monoliths are usually considered to act three dimensionally and in some cases intake/discharge monoliths are also considered to act three dimensionally. Since actual three-dimensional modeling is not a common practice for most designers, analysis of three-dimensional monoliths may be done by performing several two-dimensional analyses which capture and envelop the three-dimensional behavior.

b. Typically, a three-dimensional analysis will be performed using finite elements. A three-dimensional finite element analysis of any structure is a complicated technique. Caution is required when performing a three-dimensional analysis, and it should be performed only by an engineer who is familiar with finite elements and with the behavior of the structure being analyzed. In most cases a three-dimensional finite element analysis is not required since reasonable results can be obtained through several two-dimensional approximations. However, these approximations also require structural design experience, judgment, and insight.

c. If a frame analysis is used for analysis of a three-dimensional monolith, strips are modeled using a plane frame with in-plane loads and shear loads transferred from adjacent monoliths/strips. Strips in a miter gate monolith exhibit three-dimensional behavior due to increasing bearing pressure towards the downstream end of the monolith. The increase is due to vertical shears transferred between strips (see paragraph 7-3*b*). Accounting for the shear transferred between the strips in the two-dimensional model is essential to obtain stresses which can be compared with three-dimensional results. If the shear transfer is not properly accounted for within the two-dimensional model, then it is likely that the loads will be unbalanced, particularly if the foundation pressures or pile loads applied were obtained from a three-dimensional analysis.

7-3. Frame analysis.

a. *General*. Frame analysis is the most widely used engineering tool for analyzing U-frame locks due to its ease and speed of use. Most lock monoliths have complicated geometry, but can be modeled as a linear elastic plane frame with the use of simplifying assumptions. The frames are analyzed using CWFRAME (Jordan and Dawkins 1990), CFRAME (Hartman and Jobst 1983), or other programs. Typically a representative strip is determined for analysis.

b. *Strip selection*.

(1) For pile-founded U-frame locks, strip selection should consider pile spacing, layout pattern, stiffness, and batter. For soil-founded locks, a

1-ft-wide strip is usually sufficient. Strip selection should also consider blockouts that cause discontinuity or member property reduction in the structural framing. Some strips will have tributary load. An example of these strips/sections occurs at the culvert valve well. The walls of the well can be assumed to act as a plate fixed against rotation on three sides with reactions on upstream and downstream sections and the base slab. Wall reactions are distributed throughout the width of the upstream and downstream sections. Vertical forces are assumed to be resisted by the foundation below the wall with no transfer to adjacent sections. The wall plate must be designed to transfer the load in the assumed direction (see Figure A-4).

(2) All strips must be in static equilibrium. Strips in three-dimensional monoliths will have unbalanced vertical loads due to the foundation pressure gradient. Strip equilibrium is achieved by using vertical shears between strips with magnitudes as required to satisfy external equilibrium. This is referred to as shear transfer. Shears should be applied such that moments are not introduced into the external stability of the monolith. Shear transfer also provides redundancy in the monolith which is required to distribute the effects of small discontinuities (from blockouts/voids) in the structural framing. This permits the designer to ignore small voids in the frame analysis. Generally within a monolith, thick reinforced concrete members establish shear transfer without special details.

(3) A monolith may require several strip analyses and parametric studies in order for the designer to understand its behavior. Selecting a strip and interpretation of analysis results are challenging tasks and should be assigned to more experienced engineers.

c. Frame member. Generally, framing is modeled along member centerlines except for the very deep member that forms the culvert roof in monoliths such as the gate monoliths. This member is modeled near the top of the culvert and intersects the culvert walls at their centerlines (see Figure A-4). The block above the culvert can be modeled as a rigid body if its span-to-depth ratio is 1 to 1 or lower. If the member above the culvert is relatively thin, it behaves like a typical frame member. Member section properties are computed using member gross concrete dimensions.

d. Rigid links. Rigid links are short members at joints that are stiffened to represent the behavior of wide supports. They are used to approximate real behavior at the intersections of thick concrete members. The length of the rigid link is generally half the distance between the joint and the face of the supporting concrete. The length can be extended by half the length of a fillet if present. A link should have a stiffness of at least ten times greater than that of the intersecting flexible member. In regions of complex geometry, finite element runs can be used to calibrate the length of rigid links. For application of rigid links, see Figure A-5.

e. Foundation modeling.

(1) The foundation can usually be modeled by elastic springs (both vertical and horizontal) for soil or piles. Pile springs are attached to the base slab centerline by rigid links which model the eccentricity to the foundation. The length of the link is generally half the thickness of the base slab.

(2) Hydrostatic uplift is modeled as a load. Hydrostatic uplift reduces bearing pressures, which affects frame response.

(3) For soil-founded locks, assumed shapes of bearing pressures can be modeled as a load in lieu of using foundation springs (see paragraph 6-3*a*). Supports are still required to provide stable boundary conditions, but each support reaction should be zero. Note that displacements of the soil must be compatible with the deflections of the structure in order to accurately model the soil-structure interaction (see paragraph 6-3*b*). The use of foundation springs accommodates this requirement and is the preferred method of analysis.

(4) Pile foundation analyses should include horizontal base shears, particularly if battered piles are present. Horizontal base shear on battered piles creates a vertical component of force that will load the U-frame. Torsional moments on a monolith create horizontal base shears and should be evaluated.

(5) Due to uneven distributions of foundation bearing pressures, differential settlement between monoliths and within a monolith should be considered. Within a monolith, usually the base slab is constructed first, the subsequent load from buildup of

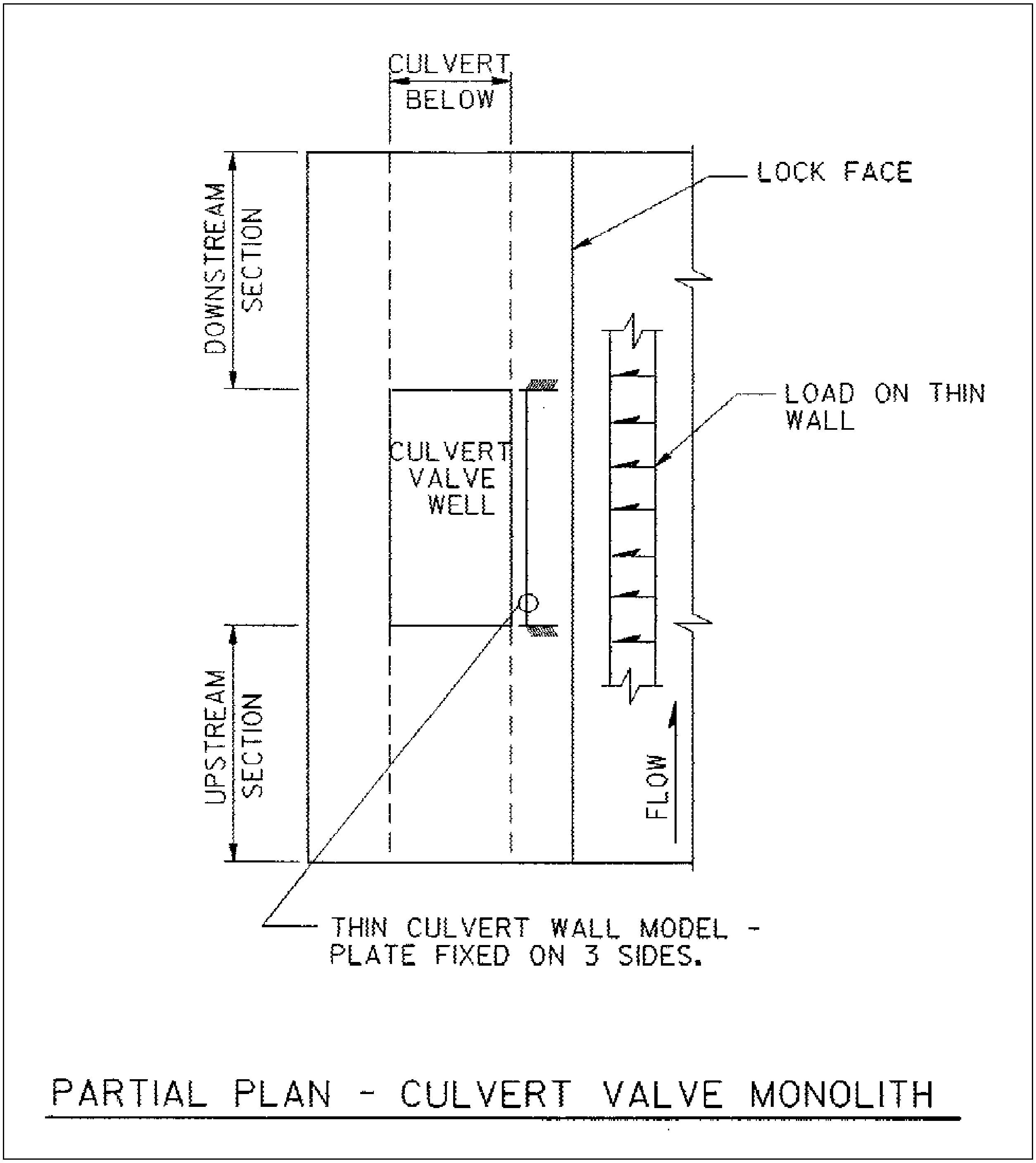


Figure A-4. Culvert valve monolith strip section (Continued)

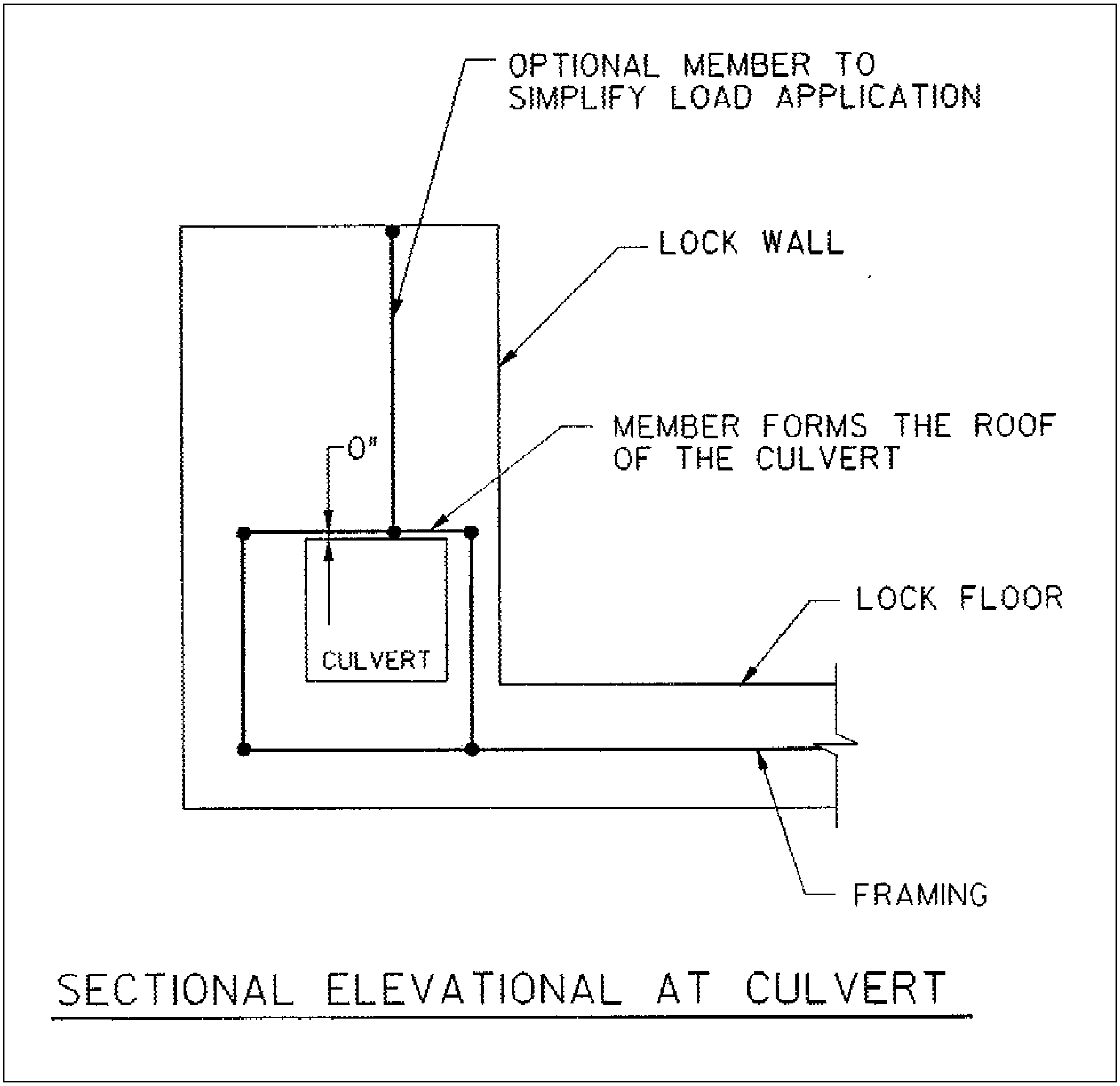


Figure A-4. (Concluded)

the walls may locally compress the foundation which will induce moments into the base slab. If the walls are constructed before the slab, generally only long-term differential settlement between the walls and slab needs to be considered. The second case creates a vertical construction joint at the intersection of the wall and base slab and should be used with discretion. This is a region of large shear and moment; therefore, the construction joint could be moved to a region of lower shear and moment as determined by analysis. Reinforcement splices must be coordinated with the location of the construction joint. Structural performance (shear-friction and diagonal-tension shear) and related detailing of the vertical joint should be carefully considered. In all cases, design assumptions should be consistent with the method of construction.

f. Variable thickness slabs.

(1) Base slabs may vary in thickness. The added concrete thickness can be designed and detailed to act compositely with the rest of the base slab. Items to ensure composite action are good construction joint

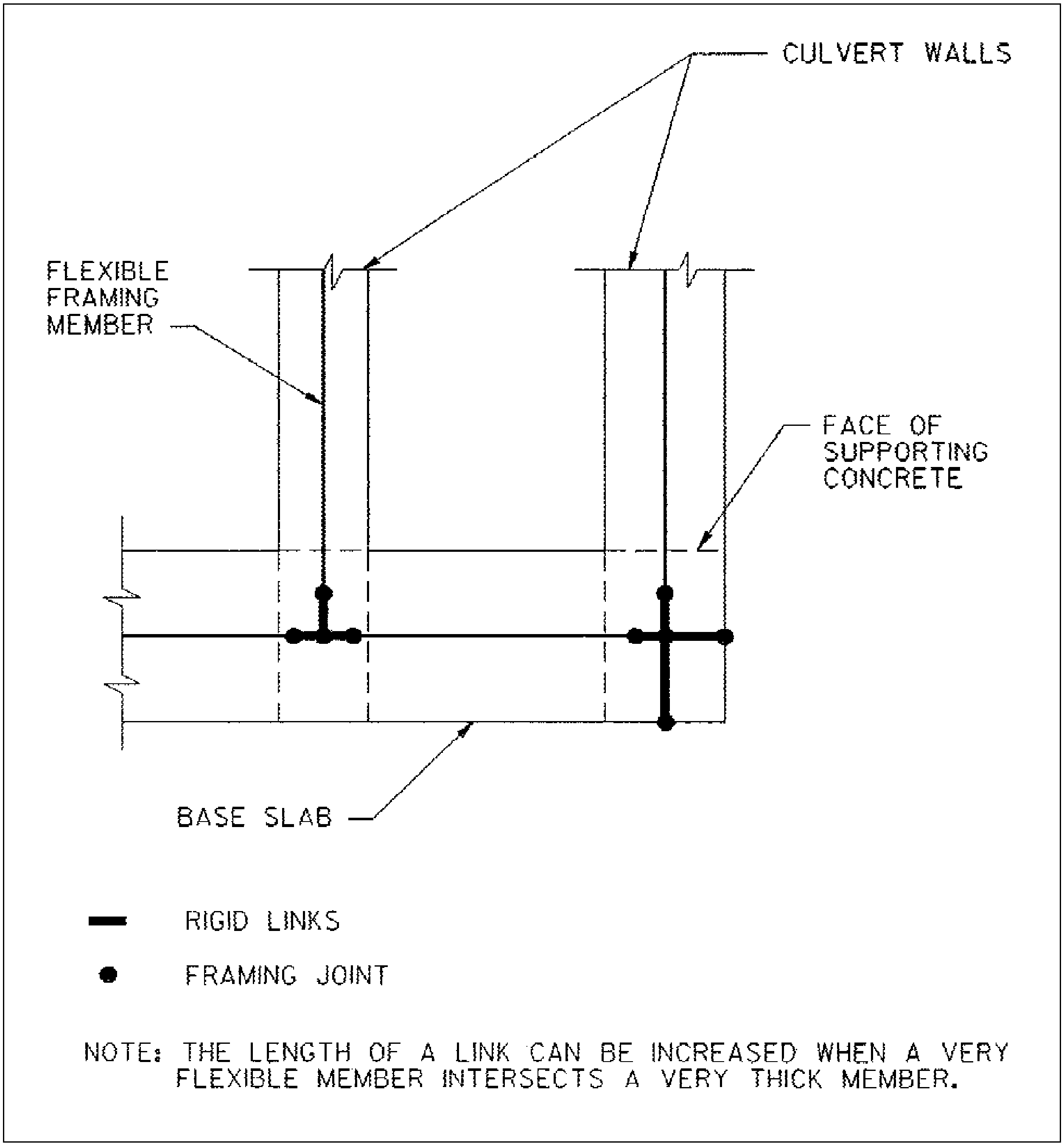


Figure A-5. Rigid link application

preparation, design for shear flow at the joint, and an adequate length of thickened section along the base member (see Figure A-6). Some thickened sections will not have sufficient length to stiffen the base slab (see related discussion on cover plated steel beams in the AISC *Manual of Steel Construction*). Alternatively, the added concrete thickness can be detailed to act independently of the rest of the base slab by segmenting it with watertight joints parallel to flow. Joint spacing should be such that composite action is not developed.

(2) Thicker slabs can stiffen adjacent thinner slabs similar to the stiffening effect that a T-beam

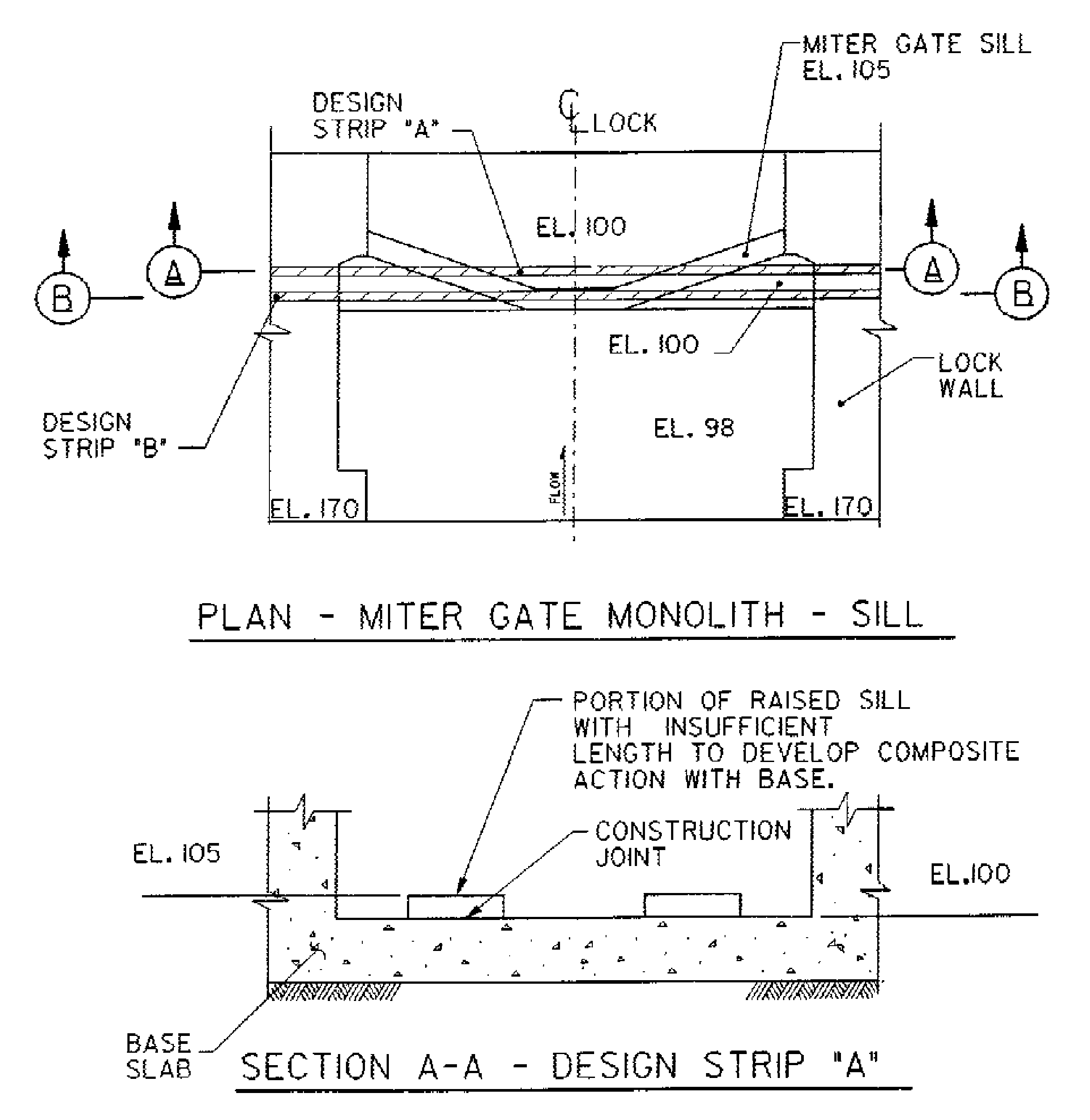
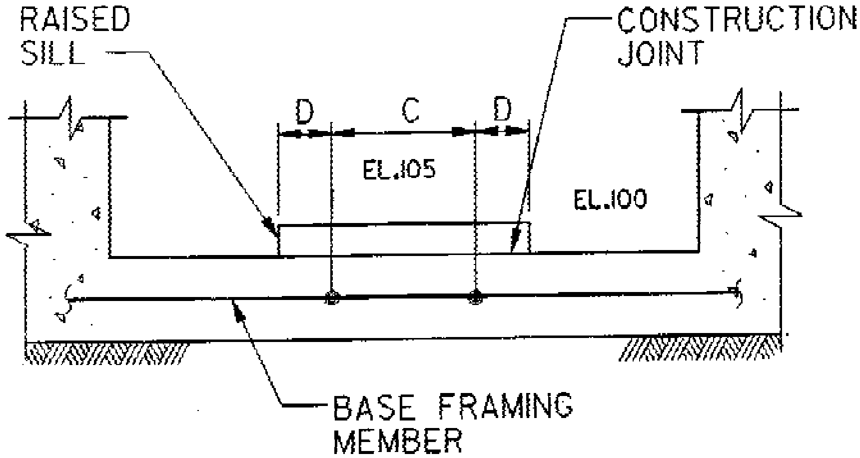


Figure A-6. Variable thickness slab (Continued)

NOTE: WATERTIGHT VERTICAL JOINTS IN SILL WILL MAKE IT INDEPENDENT OF THE BASE FRAME.



DIMENSION C - SILL LENGTH COMPOSITE WITH BASE SLAB.

DIMENSION D - LENGTH TO DEVELOP COMPOSITE ACTION. NOTE: LENGTH IS RELATED TO DEVELOPMENT LENGTH OF FLEXURAL REINFORCEMENT IN THE THICKENED SECTION.

CONSTRUCTION JOINT - SHEAR FLOW AT THE CONSTRUCTION JOINT MUST BE ADDRESSED.

BASE FRAMING MEMBER - WITHIN DIMENSION "C", BASE FRAMING MEMBER WILL HAVE INCREASED MOMENT OF INERTIA. MEMBER IS MODELED AT ℄ OF THINNER PORTION OF THE BASE.

SECTION B-B - DESIGN STRIP "B"

Figure A-6. (Concluded)

stem has on its flange. The stiffened width of the thinner slab depends on the specific geometry of the base slab and the designer's assumptions.

g. Location of design moments and shears. Joint flexural design moments at the face of supporting concrete or toe of a fillet should be used. This is also where reinforcement development length starts. Moments at these points can be found easily by modeling a joint at the desired location. Maximum midspan moments are available from computer runs or can be found using free-body diagrams. Shear should be checked, initially, at the face of the supporting concrete. Alternative locations for checking shear, in compliance with American Concrete Institute (ACI) 318, can be used if required.

7-4. Finite element analysis.

a. General. Finite element analysis is a numerical method which can be used to determine the stresses, strains, and displacements. Finite element analysis can be performed in two or three dimensions. While the finite element method is a powerful and useful tool, it must be used with care. Results from finite element analysis often appear accurate even when the input is incorrect. The apparent accuracy of finite element results stems from the fact that results are often given to the fourth or fifth decimal place. To ensure results that are accurate, it is imperative that finite element input data be thoroughly reviewed prior to proceeding with a design. Numerous texts are available on the subject and guidance for modeling with finite elements is provided in ETL 1110-2-332. ETL 1110-2-332 should be reviewed by any designer who will be performing finite element analysis. In addition, for the designer not familiar with finite elements, review of Will et al. (1987) is an excellent example of how a novice should approach finite element modeling. Finally, ETL 1110-2-254 should be referenced for the purpose of documenting finite element results.

b. Strip selection. Information on strip selection for two-dimensional monoliths can be found in paragraph 7-3*b* on strip selection for frames.

c. Boundary conditions. Boundary conditions become very important when using finite elements. In the case of symmetrical structures, boundary conditions can be used to reduce the amount of input and output produced by an analysis. This is shown in Figure A-7 where the structure can be modeled with half as many elements by taking advantage of symmetry. Boundary conditions can be used so that only a portion of the structure, such as a single wall or a portion of a wall, needs to be modeled as opposed to the entire structure, once again reducing the input and output required (see Figure A-8). This aspect becomes very important when performing two-dimensional analyses on portions of a three-dimensional monolith. The designer should use care in the selection of the applied boundary conditions so that behavior of the model represents similar behavior of the real structure.

d. Foundation modeling. Modeling of the foundation can typically be accomplished through the use of elastic springs which may be computed from the pile stiffness coefficients. Some finite element programs contain pile elements which may be used or it may be possible to employ a soil-structure interaction (SSI) model. Close coordination with the geotechnical engineer is required when selecting the spring constants for the foundation, whether it be a pile-founded or a soil-founded structure. A pile-founded structure should be analyzed so that the piles carry the entire load. If an SSI analysis is being performed, then provisions must be made so that the piles carry the load. Finite element modeling of the foundation is an option. In many cases it is not used because the increased accuracy of the results is not improved enough to justify the increased cost of the analysis. If the foundation is modeled, guidelines for developing the foundation mesh can be found in Jones and Foster (in preparation).

e. Variable thickness slabs. Typically, variable thickness slabs occur only in gate monoliths, and the shapes that result in these monoliths can be very diverse as seen by the cross sections shown in Figure A-9. Because of the shape of the base and because the loading on a miter gate monolith is three dimensional, a three-dimensional finite element analysis will likely provide the best available solution for a variable thickness slab. Due to the fact that a three-dimensional analysis is a difficult procedure, even for experienced finite element users, simplified two-dimensional finite element analyses may be used to design a base slab. Two-dimensional models which may be used to model a base slab using shell elements are the sloped plate model, the stepped plate model, or the offset beams model. These models are described in the paragraphs below. Any of these models may be used by the designer. Selection of the best method may depend on the specific geometry

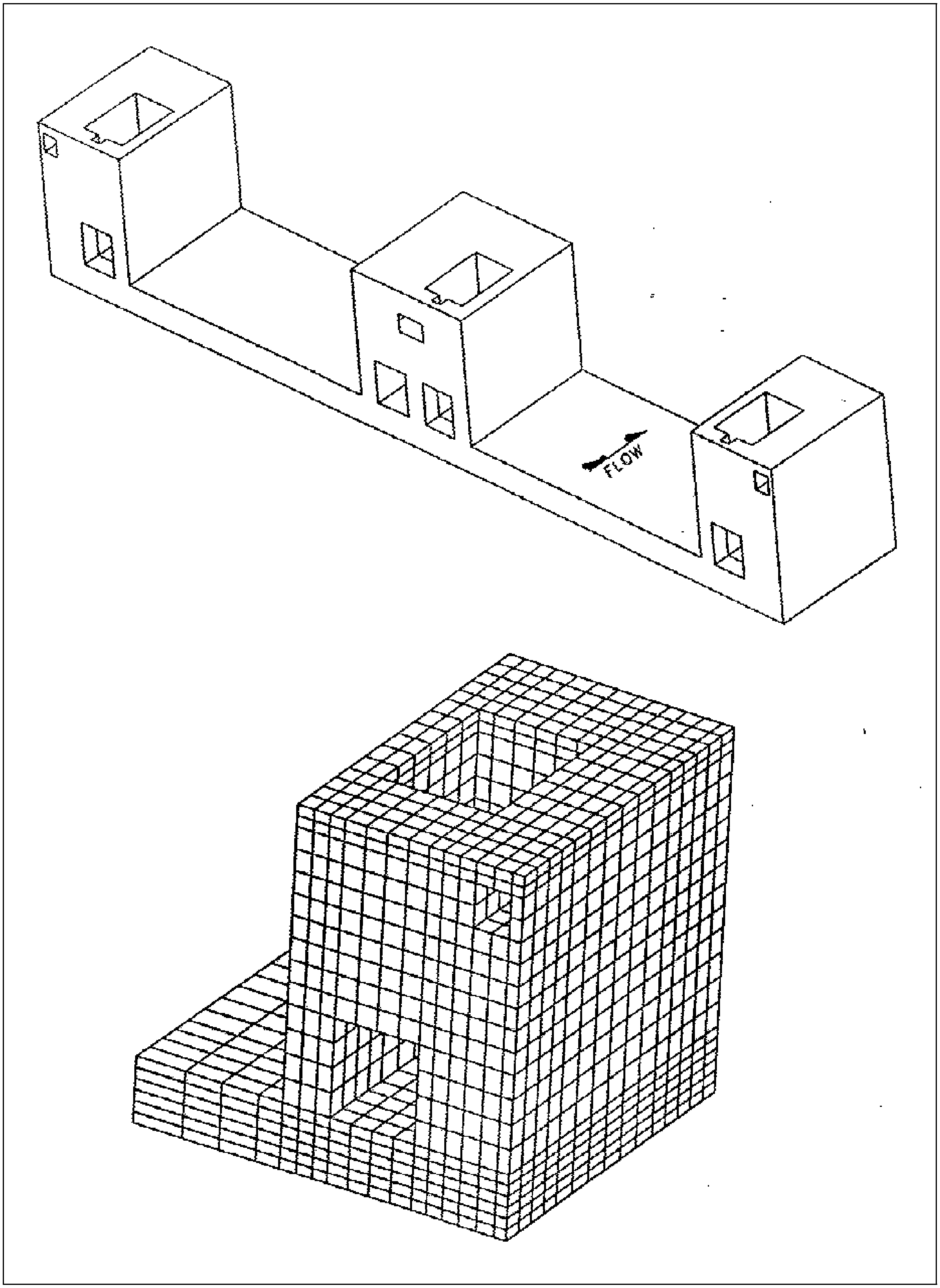


Figure A-7. Use of symmetry in finite element modeling

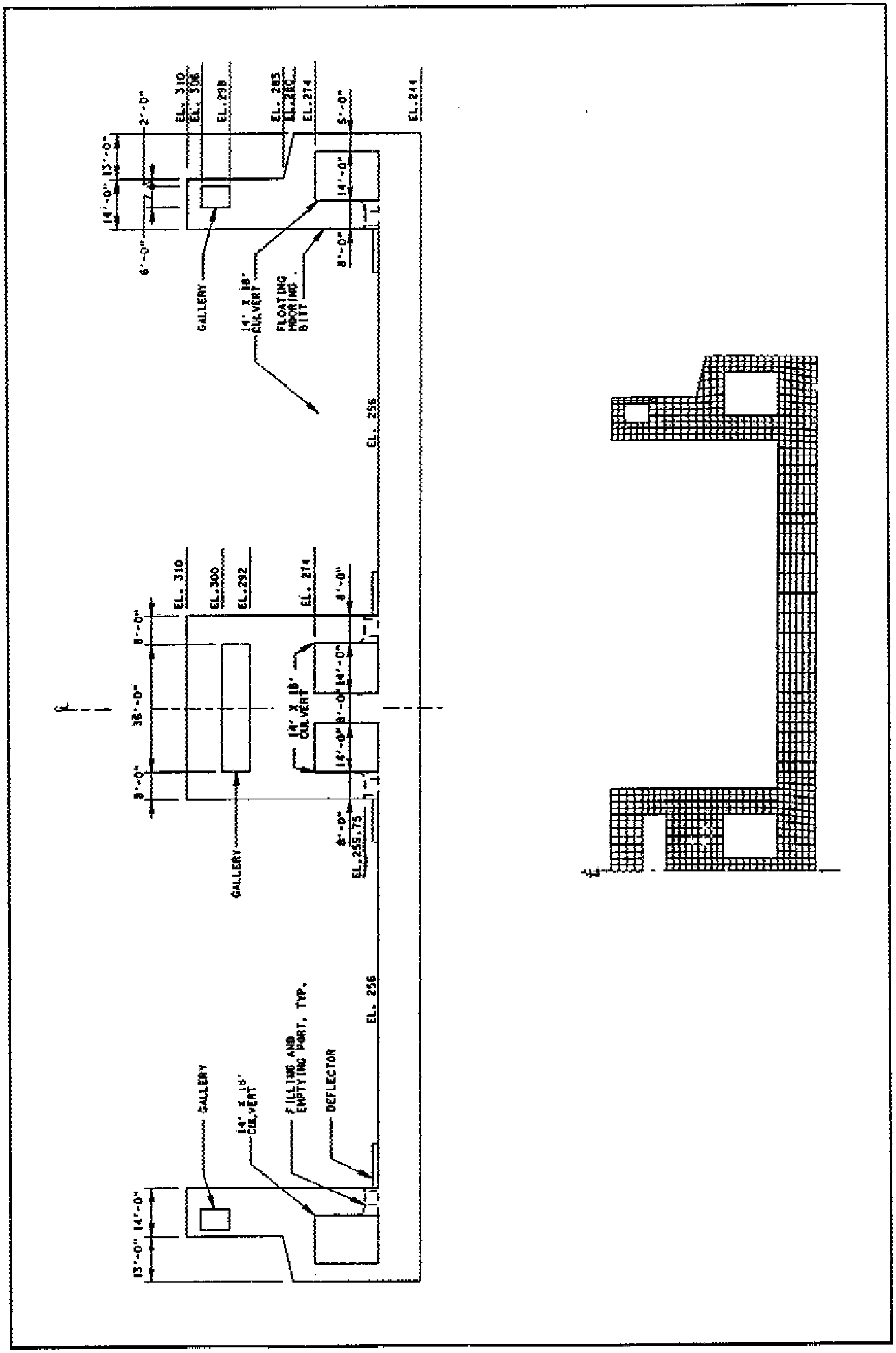


Figure A-8. Use of boundary condition to model a portion of the structure

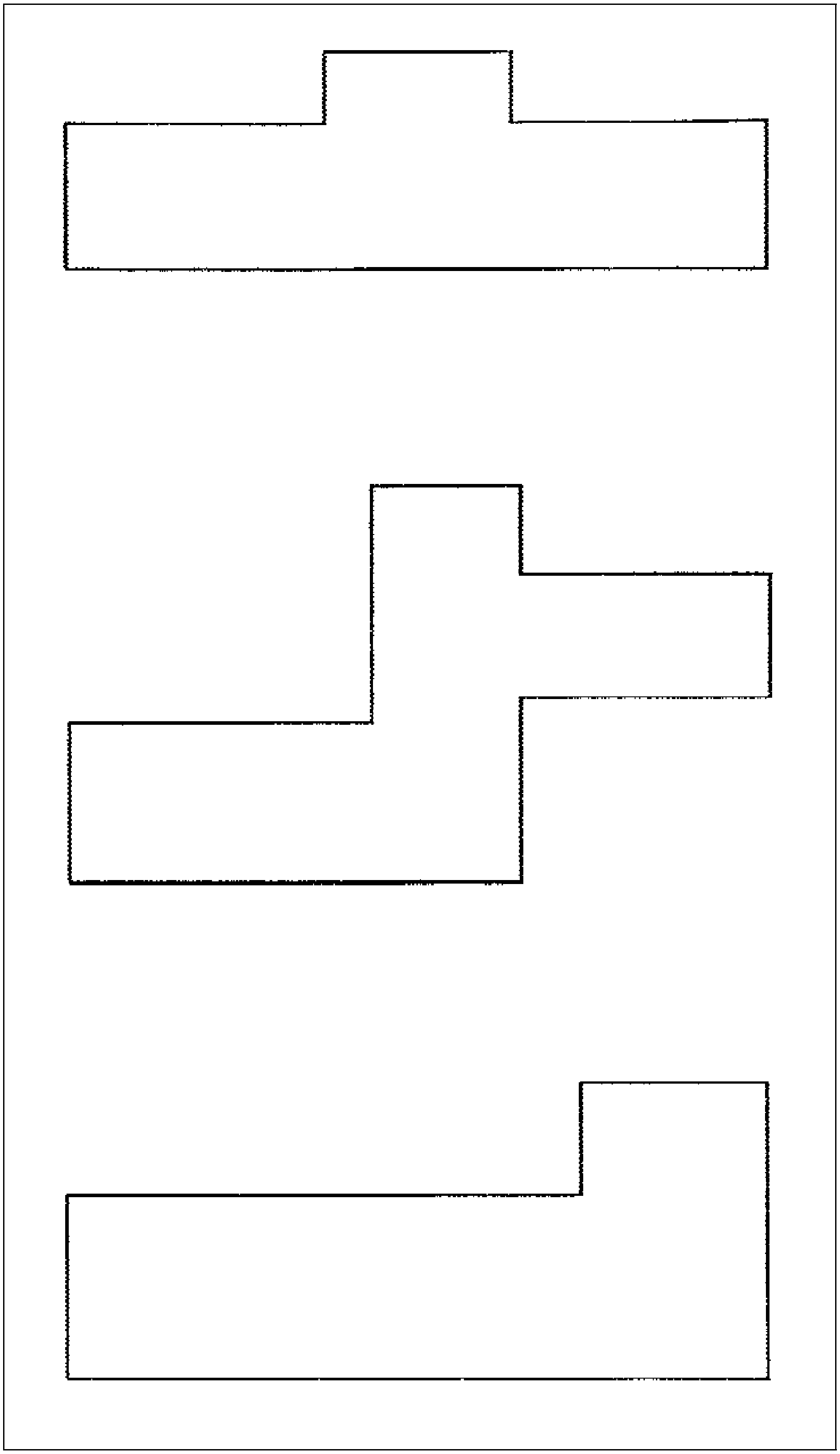


Figure A-9. Possible cross-section shapes of variable thickness slabs of the slab. In using any of the methodologies, along with engineering judgment, caution should be exercised when evaluating the results. In addition, the designer may wish to evaluate a variable thickness slab using other methods, including multiple two-dimensional strips through both transverse and longitudinal sections.

(1) Sloped plate model. The sloped plate model uses shell elements located at the centroids of the base slab. Since the centroid of the thick portion of the slab is at a different location than the thin portion of the slab, a transition between the two planes of elements is needed. This is accomplished by connecting the two planes of elements with the first row of elements in the thick portion of the base as seen in Figure A-10. A disadvantage to using this model is that the transition element is oriented unrealistically; therefore, results near this row of elements will be unreliable.

(2) Stepped plate model. Again, elements are placed at the centroids of the base slab as in the sloped plate model and again require a transition between the two planes of elements. As seen in Figure A-11, a set of vertical beams connects the planes of elements which must transmit the forces between the slab sections without introducing any unrealistic stiffness to the model. The beams may be assigned arbitrary large values for the required section properties.

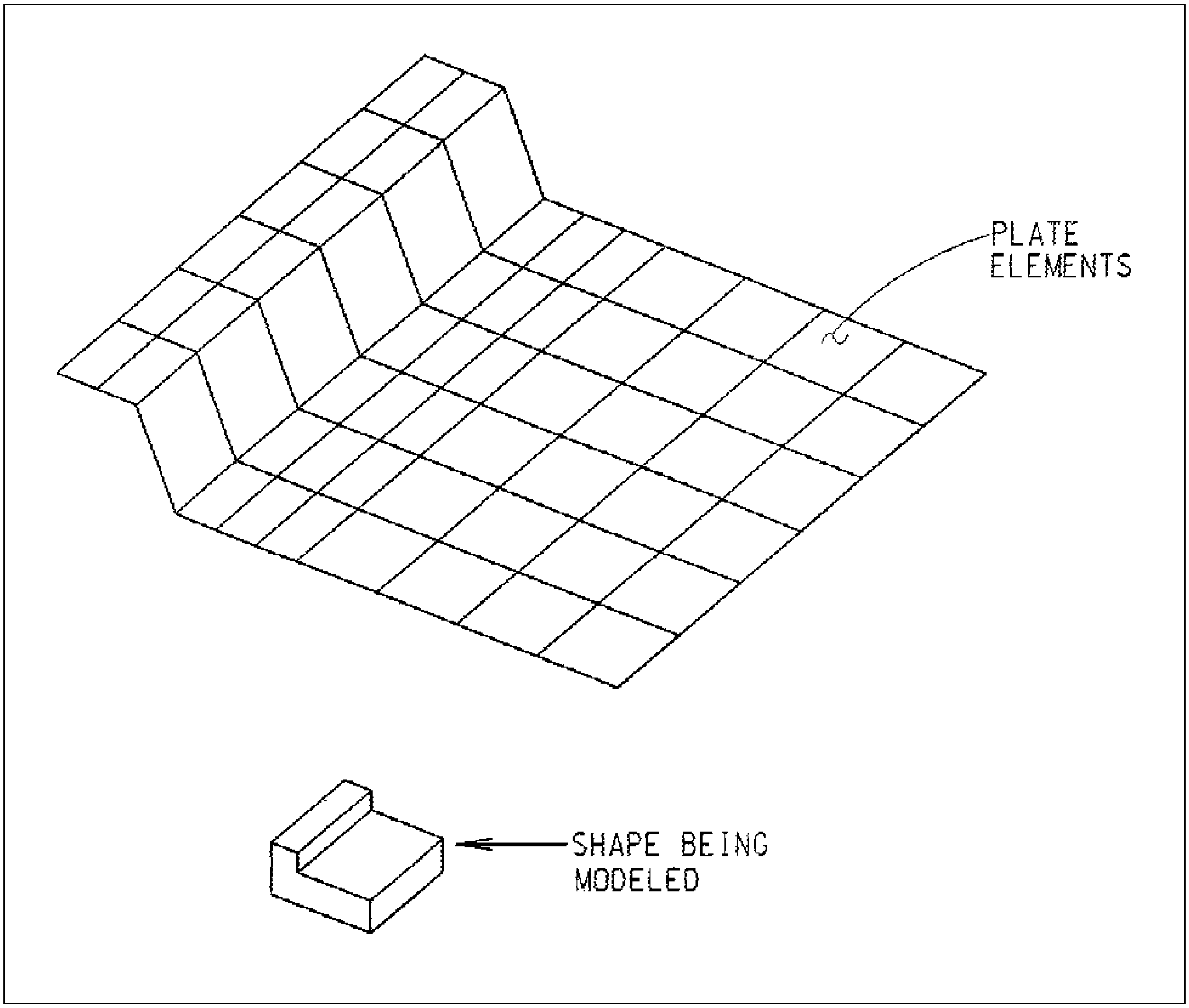


Figure A-10. Sloped plate model

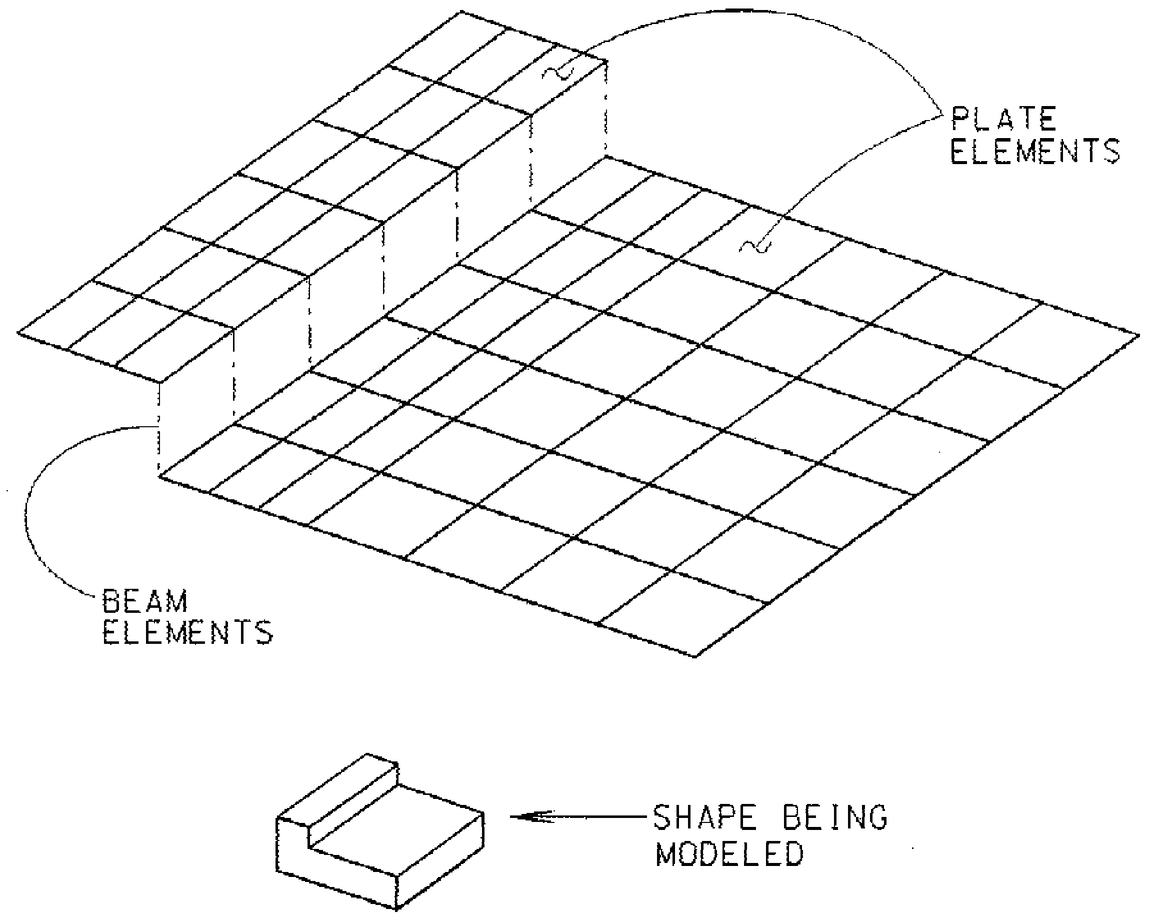


Figure A-11. Stepped plate model

(3) Offset beams model. The elements for this model are all placed at the centroid of the thin section of the base slab. At the location where the slab is thicker, a grid of beams is added. The locations of the elements and beams are shown in Figure A-12. The elevation of these beams should be located at the centroid of the additional thickness being modeled, and the section properties should also be computed based on the tributary area of the additional thickness. In order to use this model, the finite code being used must be capable of offsetting the location of the beams through a member eccentricity command since the beams are located at the same nodes as the plates.

(4) Single centroid model. The base slab may be modeled using shell elements and assigning the elements within the model with different thicknesses. This will require some approximation since the location of the centroids of the various portions of the slab will need to be placed at the same elevation, when in fact they are at different elevations.

f. Shear, moments, and thrusts (CSMT). Since output supplied by finite element programs is often in the form of stresses and displacements, steps must be taken to convert the resulting stresses into moments, axial thrusts, and shears which can used for design. To assist in obtaining the necessary shears, moments, and thrusts needed from a finite element analysis, the program CSMT was developed. The program is documented in Huff et al. (1988). The user inputs stresses along a given line from the finite element analysis into the CSMT program, and the program computes the thrust and moment from the axial stress block as well as the resulting shear on the section from the shear stress block. If the designer chooses,

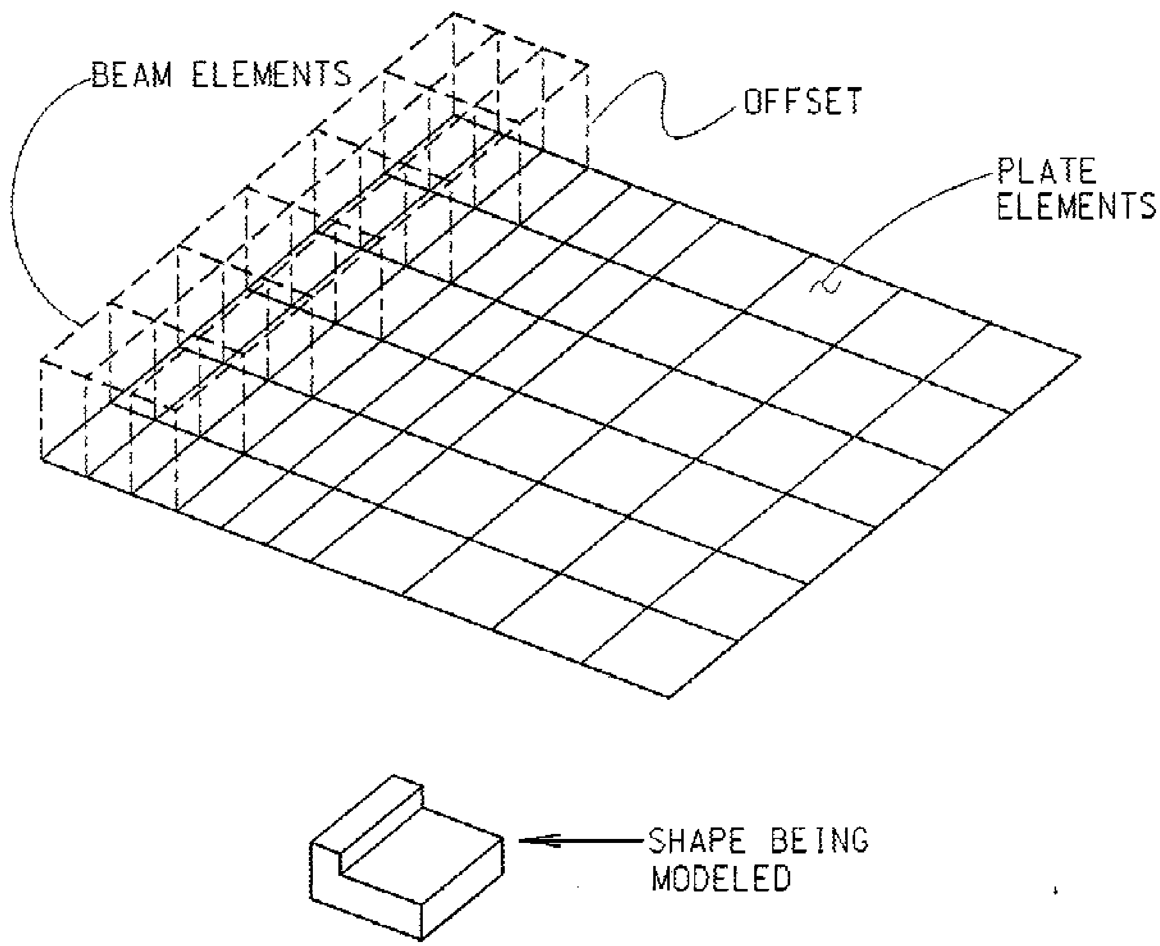


Figure A-12. Offset beams model

these types of calculations can also be performed by hand, although this can become very tedious due to the volume of data available.

7-5. Nonlinear, incremental structural analysis (NISA). A NISA is a finite element analysis which models the construction sequence of a concrete structure from the time when its first lift is placed up through when service loads are applied. Within that time frame an analysis provides results which consider the changes in concrete temperature due to heat of hydration and to ambient conditions, the placement and removal of forms, the aging modulus of elasticity, creep, and shrinkage. Properties are defined as a function of time, and the structure is incrementally constructed in the finite element model, simulating actual construction of the monolith. Guidance for performing a NISA is contained in ETL 1110-2-324. ETL 1110-2-324 requires a NISA to be performed on new types of massive concrete structures, structures which have exhibited unsatisfactory past performance, and when cost savings can be achieved through the performance of a NISA. For the design of a U-frame lock the major objective for performing a NISA would be to achieve savings through more cost-effective construction procedures and concrete mixtures.

7-6. Seismic analysis.

a. Guidance. ER 1110-2-1806 mandates seismic design considerations for all Corps of Engineers civil works projects and provides general guidance and direction for seismic design and evaluation.

Seismic analysis can be accomplished in static (pseudostatic) or dynamic terms. Dynamic analysis methods can be separated into two types: response spectrum analysis (RSA) and time history analysis (THA). Generally, initial designs should be based on pseudostatic analyses. Depending on seismological recommendations, a RSA may be required during the preparation of the design memorandums. A THA may not be required and should not be done on a preliminary basis, but may be chosen to be performed during the preparation of the design memorandums. With computational capabilities improving constantly, THA has become less formidable and can yield more efficient designs, especially when used in combination with a complete soil structure interaction model.

b. Geological and seismological investigations. The first step in performing a dynamic analysis is to obtain potential ground motion response through a geological and seismological evaluation at the site. This may yield actual site-specific motions or synthetic motions generated analytically based on site-specific geological data. Specific results from these investigations should yield definitions of the operational basis earthquake (OBE) and the maximum credible earthquake (MCE) in terms of the peak ground acceleration (PGA). Additionally, actual or synthetic (or both) time histories and corresponding response spectra should be obtained.

c. Miscellaneous considerations.

(1) Damping. Foundation and structural damping coefficients are described in other guidance. There are normally different damping values used for the OBE and MCE conditions.

(2) Backfill modeling. Modeling of backfill on lock walls is a complex issue. Generally, in pseudostatic simplified analyses, traditional lateral earth coefficient methods are used to compute backfill forces. For finite element models, modeling is normally accomplished with linear springs attached to the structure with stiffnesses based on the calculation of dynamic or pseudostatic backfill pressure. However, in actuality, during an earthquake motion the earth pressure coefficients are varying from passive to active values. Analytical models do not normally have the capacity for nonlinear springs, or if they do, they are analytically complex and computationally expensive. Therefore, engineering judgment on the value of the spring coefficients is required and must be critically evaluated throughout the dynamic analysis process.

(3) Water modeling. Water loads due to seismic forces are normally modeled in dynamic analyses relative to the Westergaard formulation of hydrodynamic forces. In pseudostatic analyses the hydrodynamic distribution of pressure can be applied as a distributed static load. In dynamic analyses this distribution is applied usually through the use of added mass attached at node locations along the perimeter of the water location. Sloshing must be taken into account. Usually calculated using Housner's method, sloshing is applied as a static load in pseudostatic analyses and as an added mass in dynamic analyses. Hydrodynamic water loads also affect the miter gate reaction loadings on the walls.

8. Special Considerations

8-1. Monolith joints.

a. Independent monoliths. Generally, U-frame lock monoliths are designed to act independently. Isolation simplifies the analysis and is a reliable basis for predicting performance.

b. Interacting monoliths.

(1) For pile-founded locks, it may be necessary for adjacent monoliths to act together to resist applied lateral loads. For example, resistance to the thrust on miter gate monoliths could be supplemented by adjacent monoliths through the use of proper joint detailing (see paragraph 9-7).

(2) For soil-founded locks, it may be necessary to key or dowel the monoliths together to minimize differential settlement.

c. Adjacent structures.

(1) Details of connections and transmitted loads from adjacent structures must be thoroughly investigated. Poor detailing at these connections could result in localized failures and/or serviceability problems. Some areas to look at are: cofferdam tie-ins, abutting dam piers and their joint treatment, and guidewall tie-ins. The load for designing sheet pile tie-in connections should consider the interlock force of the piles as referenced in EM 1110-2-2503.

Watertight joints to adjacent dam piers will change the hydrostatic loads applied to the lock.

(2) Adequate joint thicknesses between structures permit them to maintain separation when deflecting under load. Guidewalls can have either a dead load reaction or impact reaction on the lock.

8-2. Seismic effects.

a. In general, a U-frame lock is inherently seismic resistant due to the lock symmetry (no torsion), monolithic shear wall and diaphragm action in void areas, integral foundation mat, wall aspect ratio, and overall rigidity. All of the above are benefits achieved by good geometrical layout and proper detailing of reinforcing. The main seismic weakness of a U-frame lock is the large base shears transferred to the limited foundation lateral load resisting system.

b. Structures adjacent to the lock should have sufficient separation (joint width) from the lock such that the two structures will not strike each other during a seismic event. Deflections used to size joint thickness are computed using concrete gross section properties since the members are expected to remain in the elastic range. Cracked section properties will be used if seismic analysis of the frame indicates a nonlinear response.

c. The service bridge seats must have sufficient bearing length (pinned ends) to accommodate the largest lateral movement experienced by the bridge piers combined with a reasonable assumption for the thermal contraction of the bridge. Proper detailing will ensure that the bridge remains seated during a seismic event.

d. Lateral loads above large voids must be transferred to the mass below the void and foundation using interior walls as shear walls or by frame action if shear walls do not exist (i.e., culverts and galleries). To ensure ductile frame action during seismic loading, typical ACI reinforcement details (ACI 318) should be reviewed for applicability. Contiguous concrete, perpendicular to analysis strips, provides joint confinement which will reduce spalling concrete, and therefore generally eliminate the need for special seismic detailing.

e. It is necessary to check equipment anchorage for the OBE since the lock must remain in operation during and after this event.

8-3. Effects of voids. The use of voids or blockouts is acceptable. Voids and blockouts reduce the weight of the structure as well as the loads on foundation piling and/or foundation pressures. Voids or blockouts also reduce the amount of concrete needed. Using voids will increase the effort required to design and build the monoliths, but should result in less expensive structures due to reduced concrete required in the base and wall sections. Reducing the amount of wall concrete will reduce the top base bending moments, and possibly reduce the required base thickness. A means to remove seepage water from the void must be provided or the seepage water weight must be included in the monolith design.

8-4. Foundation drains. Foundation drains beneath the lock and along the landwall of the lock are used to reduce the piezometric head from seepage from the upper pool to the lower pool. The foundation drains beneath the lock monoliths may be french drains, consisting of either select sand or select sand with filter drains. These drains are usually connected to the lower pool with no control on backflooding. Drains along the lock landwall are used to reduce the horizontal hydrostatic load acting against the lockwall. Such drains consist of horizontal runs of wellscreen or perforated pipes connected to vertical cleanouts and manholes. Means of preventing backflooding through the drains should be incorporated into the drain design. If the drain is relatively deep in relation to the height of the lockwall, it is recommended that stainless steel wellscreen and pipe be used for the horizontal drain pipe. Reducing uplift during dewatering by exiting foundation drains into the lock chamber and then removing the drainage with pumps should also be considered. The effectiveness of drains should be considered in analysis.

8-5. Instrumentation. Instrumenting the lock structure and its foundation can serve two basic functions. Site personnel can monitor performance while the lock is in service, and design assumptions and parameters can be verified. Some examples of data that can be accumulated include uplift and pore pressures, monolith tilt and alignment, cofferdam-cell movements, concrete crack widths, and internal concrete temperatures. More information on instrumentation for structures can be found in EM 1110-2-4300, and more information on foundation instrumentation can be found in EM 1110-2-1908. Many types of instrumentation exist which serve different purposes, but all forms require planning for both design and construction. Consideration of instrument installation during the construction process is crucial for the successful long-term operation of the equipment and to minimize disturbances to construction.

8-6. Silt. Silt accumulations upstream of miter gates and in the lock chamber may become an operational problem. Large accumulations of silt can restrict the free movement of the miter gates. There are several methods available for removing silt from around miter gates. Blockouts may be used upstream of the gates to provide more area for silt to accumulate in before it must be removed. A silt flush system using water may be installed to resuspend the silt in the water and move it out of the gates' path. Small-diameter pipes can be placed in the lower gate sill to provide for flushing action around the lower miter gates.

9. Constructability Considerations

9-1. General. A U-frame lock design includes details that ensure structural performance and can be accurately detailed, bid, and constructed. Good detailing will be rewarded by reducing contract modifications and reducing engineering effort when interpreting the plans for field personnel and the contractor. There are some areas that are troublesome, and these will be briefly discussed in the following paragraphs.

9-2. Construction sequence for monoliths.

a. Typically two monoliths will be constructed separated by a space for a third monolith. The middle monolith will use the end monoliths as formwork and as supports to which joint material can be attached. Expansion joint material must not be compressed by the fluid force of fresh concrete, but must be compressible to accommodate thermal expansion of the monoliths.

b. Construction sequencing of the monoliths should be to place the deeper founded monoliths first. This eliminates potential undermining and loss of foundation confining soil during excavation if the more shallow monolith were placed first. Backfilling of overexcavation for the deeper founded monoliths should be done prior to placing concrete for the adjacent monolith.

c. The general shape of a monolith's bearing pressure diagram is characteristic for differential settlement between the walls and the base slab. Concrete placement sequence also affects the differential settlement of the monolith.

9-3. Reinforcement placement. Proper reinforcement detailing will simplify congested areas, ensure ease of reinforcement placement, and facilitate the placement of concrete.

a. Congestion. (See reference ACI 309.3R for related discussion.)

(1) U-frame locks generally have small amounts of reinforcement in relationship to the volume of concrete; however, reinforcing is concentrated at the concrete faces. Reinforcement layering and bundling of bars can reduce congestion; however, additional layers of horizontal reinforcement can create difficulties in the placing of concrete and bundled bars have characteristics (ACI 318) that may make them undesirable.

(2) Possibly the most congested area of reinforcement is at the intersection of the base slab and the lock wall (particularly for culvert intake and discharge manifolds). At this location, wall dowels intersect and may conflict with the layers of base slab reinforcement. Generally, main structural reinforcement should be located first and less important reinforcement spaced to eliminate interference. For example, the placement of vertical reinforcement in the lock wall may be dependent upon missing voids or embedded items encountered higher up in the wall. It follows that base slab reinforcement should be detailed, to scale, to avoid the vertical lock wall reinforcement. Other items which may contribute to congestion are heavy structural steel for support of reinforcement, electrical conduits, instrumentation, embedded metals, heavy reinforcement requirements at ports/manifolds, and formwork supports.

(3) The contractor may opt to set the ends of wall vertical bars at the top of concrete lifts instead of holding them up at elevations as specified on the drawings. Although this is generally allowed, the additional reinforcement may add to the congestion.

(4) Congestion and other construction problems may become evident if reinforcement splices and development lengths are drawn to scale on the contract plans.

b. Splicing.

(1) In general, the use of splices should be minimized, and all contractor-added splices should be carefully reviewed to ensure that they do not adversely affect structural performance. All splice locations and splice staggers should be in compliance with the latest ACI 318 recommendations and shown on the contract plans or indicated in the specifications. In determining splice locations, the designer can use the normal maximum fabricated length of 60 ft for horizontal bars. Forty feet is usually used in detailing the maximum length for vertical reinforcement because of constructability limitations.

(2) Numerous load cases cause inflection points and regions of high moments to vary. The designer should consider this phenomenon when locating splices.

(3) U-frame locks require #14 and #18 bars to resist large bending moments. Large reinforcing bars are difficult to fabricate, ship, handle, place, and support. Large reinforcing bars also require mechanical splices. Mechanical splices are difficult to inspect, the coupler adds to the congestion, and the possibility of installation error is greater than for a lap splice. Large reinforcement bars should be used only where required.

c. Bending.

(1) Bends in reinforcement are usually made for standard hooks, corner bars, and bar terminations. Large bars have large bend radii that often interfere with the placement of intersecting steel (see Figure A-13). It is suggested that all bar bends be drawn to scale on the plan drawings to help identify and correct reinforcing placement problems. Horizontal #14 and #18 bars with bends will cause problems since they will be tied to vertical bars set in hardened concrete that may have been placed without considering the large bend radii of the horizontal bar.

As a result, concrete clearances will be sacrificed (see Figure A-14). A possible solution is to detail vertical bars along the bend of the large horizontal bars.

(2) Hooked #14 and #18 horizontal bars have long extension lengths at their free ends (3 ft-5 in. for #18 bars). If the free end protrudes a small distance from the upper lift into a lower lift, it is permissible to rotate the end from the vertical position until the free end is out of the lower lift. This allows the contractor to place the bar on top of the lower lift after it has hardened. Alternatively, the free end can remain vertical and the lower lift can be blocked out to receive it (see Figure A-15). This minimizes the number of reinforcing mats that the contractor has to support during a concrete placement. The option of rotating the free end of the hook or blocking out can be given to the contractor with reference to a note on the contract plans.

d. Reinforcement and waterstops. Waterstops will not be omitted or punctured so that the reinforcement can run its normal path. Adding concrete cover over the reinforcing bars or detailing reinforcement around the waterstop in the initial design can eliminate this problem (see Figure A-16).

e. Geometric discontinuities. At blockouts (i.e., recess in a wall), flexural reinforcement must be terminated, usually in a standard hook, and structural continuity reestablished by placing additional reinforcement under or to the sides of the blockout. Added reinforcement should be developed past each end of the blockout. Added reinforcement under the blockout probably requires widely spaced (larger than 6 in., ACI 318) noncontact lap splices. Additional reinforcing, such as development length past the potential crack zone (ACI 318) or stirrups to control cracking, may be required (see Figure A-17).

9-4. Fillets.

a. Fillets are used in U-frame locks for many reasons and in many locations. Fillets from the floor to the wall in the culvert have many advantages and disadvantages.

(1) Advantages. Fillets can reduce honeycombing in the main structural members, and permit the use of the lower design moments and shears which occur at the toe of the fillet.

(2) Disadvantages. Disadvantages include: planes of weakness in the construction joints of the fillet, suspended forms for the fillet and culvert floor

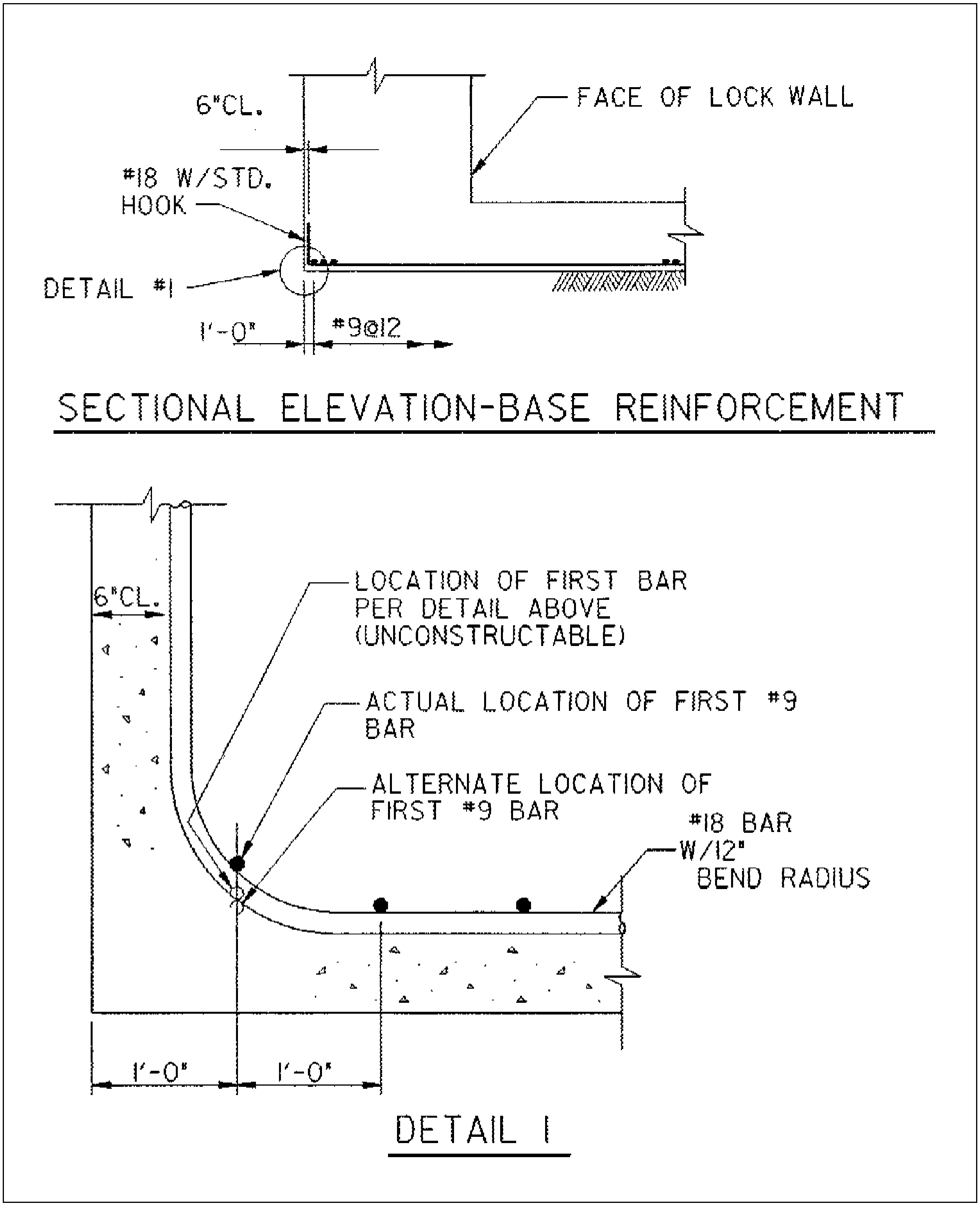


Figure A-13. Reinforcement interferences in base slabs

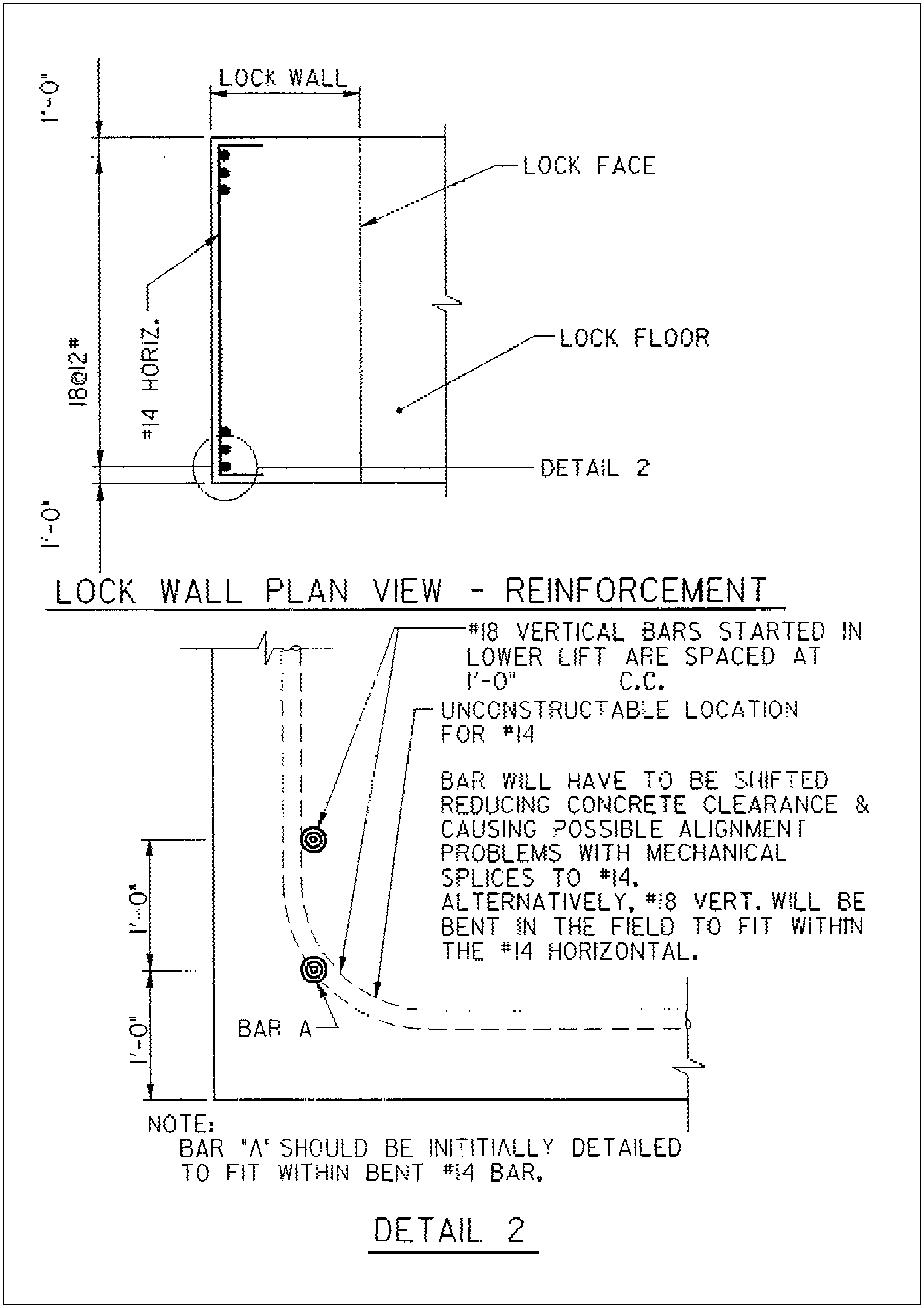


Figure A-14. Reinforcement interferences in lock walls

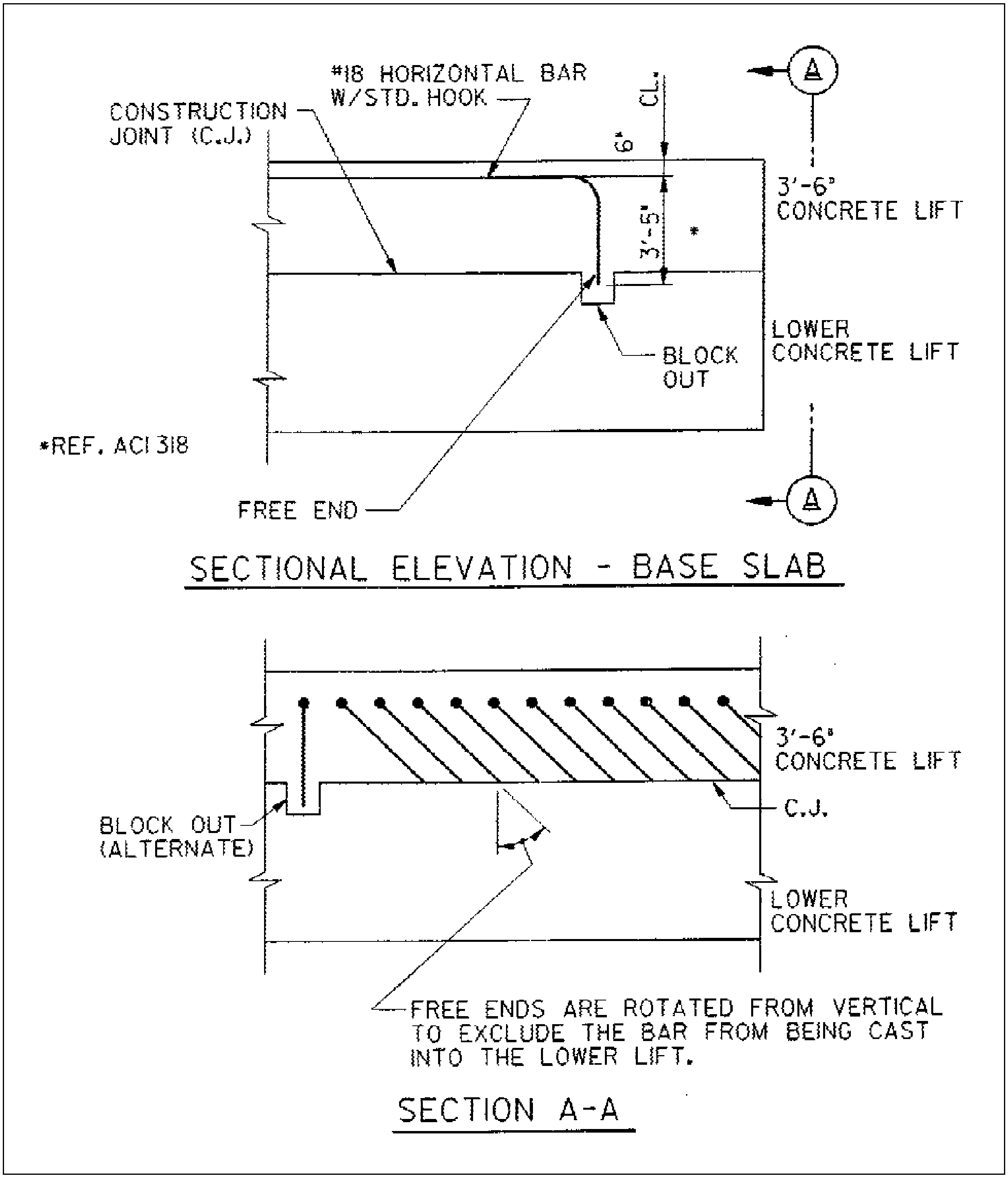


Figure A-15. Solutions for reinforcement placement problems

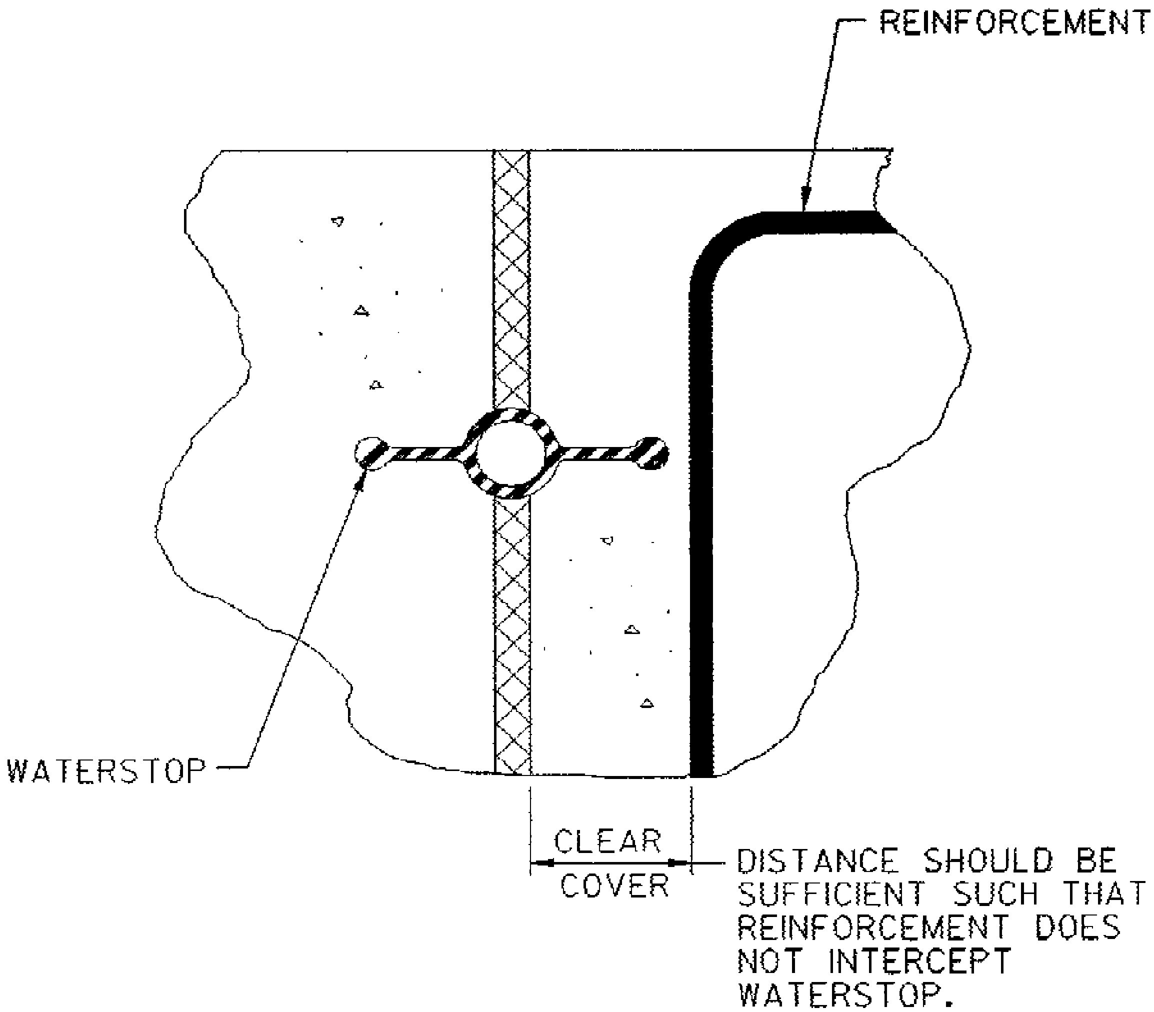


Figure A-16. Reinforcement and waterstop interferences

are required to eliminate planes of weakness, susceptible to honeycombing in the toe of the fillet, fillets are discontinuous at lock filling and emptying ports. Due to the many disadvantages, the floor-to-wall fillet is not recommended. Fillets at the roof of the culvert are less of a problem and can be advantageous.

b. The designer should detail the construction joint at the wall to the base slab at the level of the culvert invert. The joint should be cleaned, possibly roughened, and reinforced to ensure an adequate connection. This joint location also facilitates the finishing of the culvert invert and base slab (see Figure A-18).

c. In summary, the potential benefits from a fillet and its probable method of construction should be reviewed by the designer before using it in analysis and design.

9-5. Construction/lift joints. Selection of lift joints should be closely coordinated with the materials engineer and a representative of the construction division. Lift heights in base slabs of U-frame locks usually do not exceed 5 ft while lift heights in the walls are typically 5 to 10 ft high but can exceed 15 ft in the culvert walls. Typically any changes to optimize originally selected lift heights will be

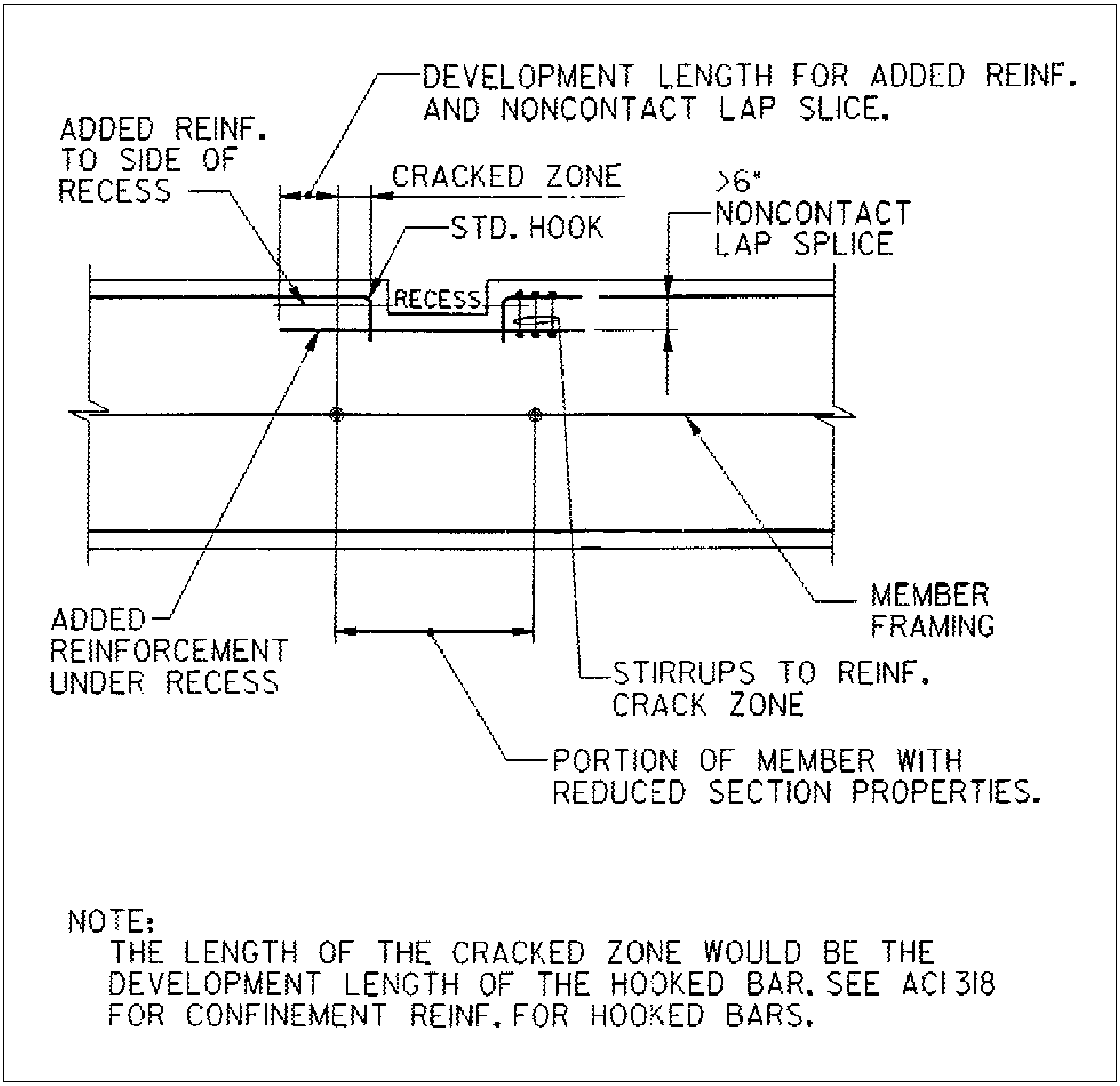


Figure A-17. Reinforcement at geometric discontinuities

determined during the course of a NISA. When considering the lift heights in the wall, a construction joint should be placed at the top and bottom of all voids. Other changes in geometry are also places where lift joints should be located. The lift heights selected should be made as consistent as possible on all monoliths to allow the contractor to use one set of forms in a number of different locations. For large slab placements where vertical construction joints are necessary, efforts should be made to locate these joints in the lowest stressed areas of the slab. In addition, if tension is expected to occur across these joints, then appropriate measures should be taken to prepare these joints with reinforcing dowels being placed across them.

9-6. Monolith joints (monolith length). The length of monoliths is determined by evaluation of constructability, temperature effects, and cost. Length of monoliths will generally range from 50 ft to over 100 ft.

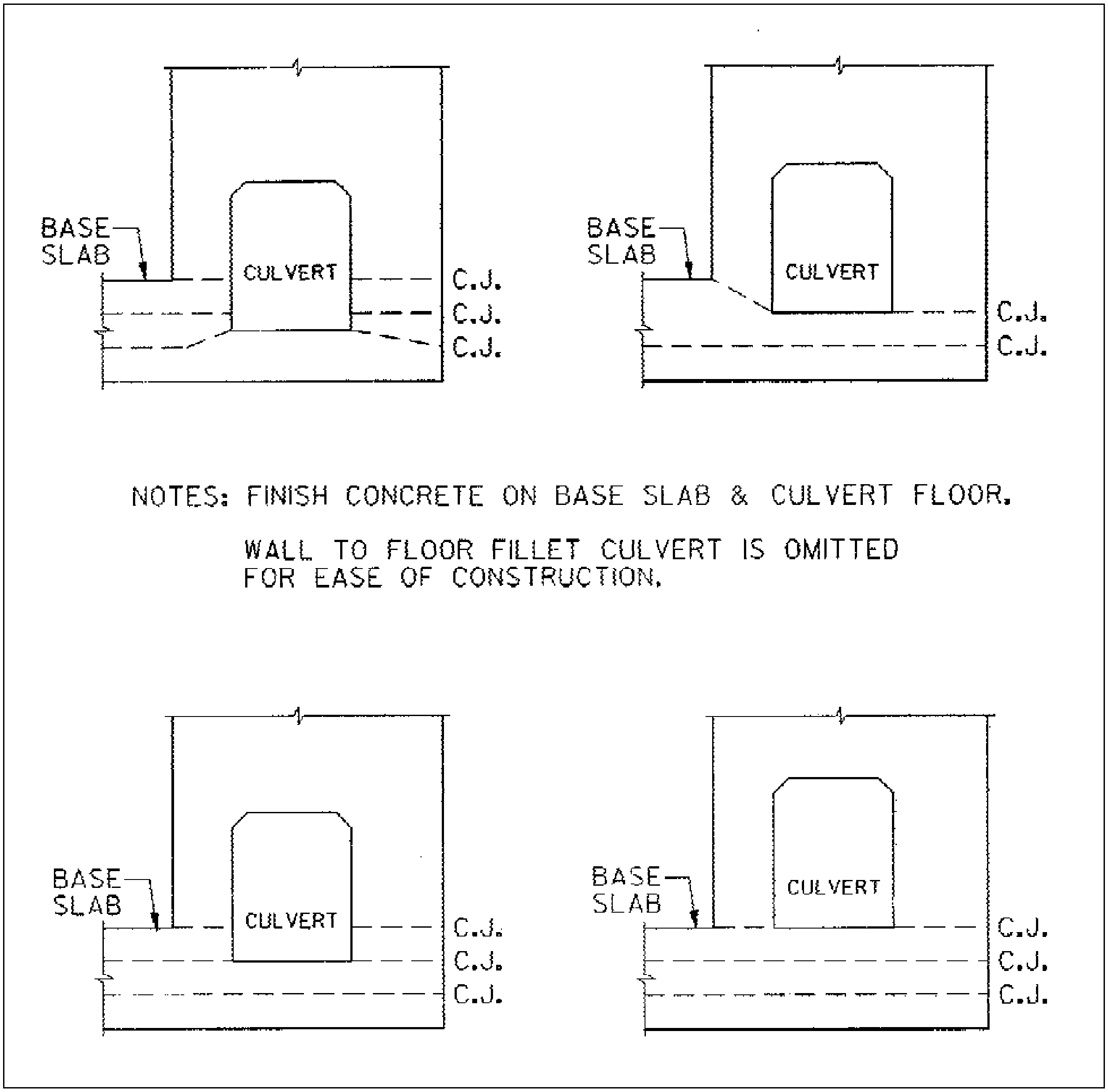


Figure A-18. Varying elevations of construction joints and concrete finish floor elevations

a. Constructability. One of the primary factors when determining monolith length is the capacity of the concrete batch plant to be used at the project. Generally this factor is resolved by the materials and construction engineers. The location of the culvert ports and their spacing must be accounted for when determining the length of a monolith. Monolith spacing should be arranged such that a monolith joint is approximately half way between culvert ports. Similarly, accommodations for instrumentation recesses and their spacing should be considered prior to finalizing the length of a monolith.

b. Temperature effects. The length of a monolith may be limited by the effects of temperature. Generally, the longer a monolith is, the higher the stresses in the longitudinal direction. Determination of a suitable monolith length with respect to temperature can typically be made from experience and the performance of a NISA.

c. *Cost*. Increasing the lengths of monoliths in a U-frame lock is generally considered to be a cost-saving measure. Increasing the length of monoliths will reduce forming costs and the number of joints to receive preparation. Another item which should be considered with respect to cost is the length of the reinforcing bars. Since reinforcing bars are generally shipped in lengths of 60 ft, a monolith slightly longer than 60 ft may not be cost effective due to splicing which would be required.

9-7. Joint treatment/detailing.

a. Independent monolith action is ensured by proper detailing at monolith joints. The joint thickness should consider the monolith deflections toward each other and the compressibility of the joint material.

b. The base slabs of adjacent monoliths may be required to act together to resist shear, tension, and/or compression (see paragraph 8-1*b*). This is done by doweling the bases together to obtain a shear-friction connection or by the use of shear keys. Tension between base slabs is resisted by dowels. Compression is resisted by concrete bearing. Joint treatment between monoliths should be compatible with the type of force(s) transmitted across the joint. A water-stop at the joint between monoliths will be required to stop the transmission of foundation material through the joint. Walls of monoliths can be detailed and analyzed to act independently if their base slabs are doweled together.

c. Further discussion on joint treatment/detailing is found in other guidance.

10. Specifications and Details

The items discussed herein address the analysis and design of U-frame lock monoliths without discussing the specifications and details required to complete a design. While not addressed specifically in this guidance, proper attention to the specifications and details is a vital part of the complete design process. Guide specifications are available for use by the designer and should be studied carefully and modified as necessary for each project. Good details are an essential element in producing a quality product. Designers should draw on past experience and review other available guidance.